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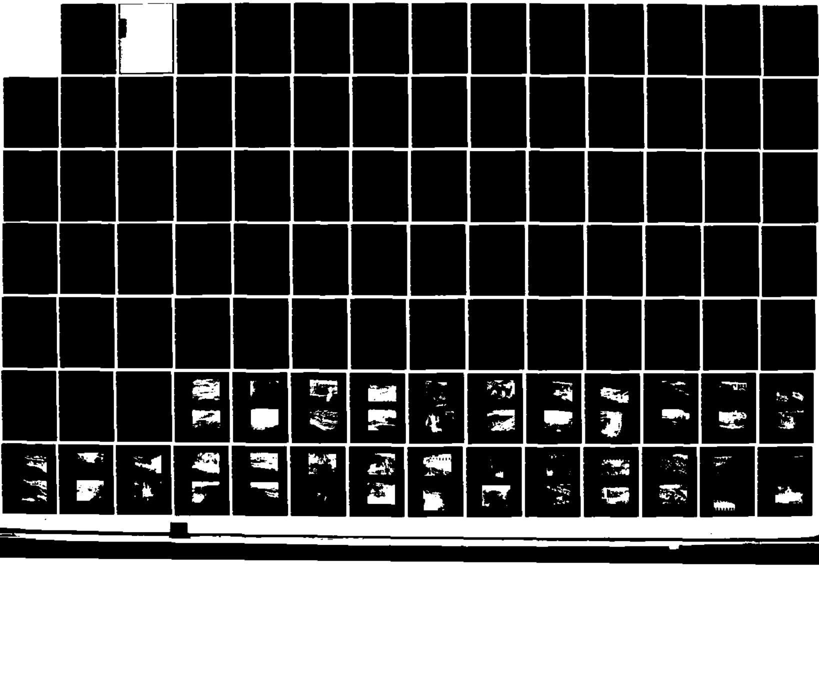

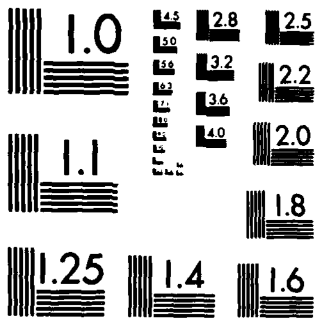

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The purpose of this report is to provide a record of foundation conditions encountered during construction and methods used to adapt to these conditions during construction. This information is a part of the permanent collection of project engineering data required by Appendix A to ER 1110-1-1801, dated Dec 81. This report deals with construction of the main embankment, the Sterett Creek Dike and the Spillway-Powerhouse.

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OPERATION AND MAINTENANCE MANUAL
HARRY S TRUMAN DAM AND RESERVOIR
OSAGE RIVER, MISSOURI

APPENDIX VII
CONSTRUCTION FOUNDATION REPORT

VOLUME ONE

1984

DEPARTMENT OF THE ARMY
KANSAS CITY DISTRICT, CORPS OF ENGINEERS
KANSAS CITY, MISSOURI

OPERATION AND MAINTENANCE MANUAL
HARRY S. TRUMAN DAM AND RESERVOIR
OSAGE RIVER, MISSOURI

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OPERATION AND MAINTENANCE MANUAL

HARRY S. TRUMAN DAM AND RESERVOIR
OSAGE RIVER, MISSOURI

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CONSTRUCTION FOUNDATION REPORT

CHAPTER 1

INTRODUCTION

1-01. Location and Description of Project: Harry S. Truman Dam (Kaysinger Bluff Dam) is located at mile 175 on the Osage River in Section 7, T. 40 N., R. 22 W., Benton County, Missouri about 1 1/2 miles northwest of Warsaw and 95 miles southeast of Kansas City. At the top of multipurpose pool, El. 706 m.s.l., the lake extends upstream beyond Osceola, Missouri on the Osage River and to the vicinity of Clinton, Missouri on the South Grand River. At the top of flood control pool, El. 739.6 m.s.l., the lake entends to within 8 miles of the Missouri-Kansas State Line. The lake is located in: Benton, Hickory, Henry, St. Clair, Vernon, and Bates Counties, Missouri. Drainage area above the dam is 11,500 square miles which includes 3,544 square miles of area controlled by other reservoirs upstream (Pomme de Terre, Stockton, Hillsdale, Pomona and Melvern). The flood control pool, El. 739.6 m.s.l., covers 209,300 acres and stores 3,000,300 acre-feet of water. The multipurpose pool, El. 706 m.s.l., covers 55,600 acres and stores 1,202,700 acre-feet of water. The dam consists of: (1) A zoned earth and rock embankment, 5,100 feet long, about 96 feet above the valley floor, and 120 feet above streambed. Crest elevation is 757 m.s.l. Volume of the embankment is about 6,200,000 cubic yards. There are three small earth dikes in saddles on the right bank bluff downstream of the main dam and a large embankment, Sterett Creek Dike (1,686,000 cubic yards) across a tributary valley on the left abutment. (2) A reinforced concrete powerhouse and gate controlled overfall spillway located near the right abutment. The powerhouse contains six, inclined, reversible, pump-turbine, motor-generators with a total rated capacity of 160,00 Kw. Water will be "pumped back" into the reservoir from Lake of The Ozarks for "pump back" storage operation. Concrete quantity is 296,625 cubic yards. The spillway outlet works has five 6.5 feet by 14 feet slide gates. The spillway is 240 feet wide with six tainter gates, each 25.7 feet by 40 feet. Spillway capacity at flood pool elevation is 200,000 c.f.s. Total estimated cost of the project (1981) is $532 million.

1-02. Construction Authority: Harry S. Truman Dam and Reservoir project was authorized by the Flood Control Act of 1954, approved 3 September 1954 (Public Law 780, 83rd Congress, Second Session). The project was modified to include power by Flood Control Act approved 23 October 1962.

1-03. Purpose and Scope: The purpose of this report is to provide a record of foundation conditions encountered during construction and methods used to adapt to these conditions during construction. This information is a part of the permanent collection of project engineering data required by Appendix A to ER 1120-2-100 and ER 1110-1-1801, dated 15 Dec 81. This report deals with construction of the main embankment, the Sterett Creek Dike and the Spillway-Powerhouse.

1-04. Project History: The embankment and Spillway-Powerhouse was constructed in six stages, spanning 13 years. Work began with the Stage I contract in September 1966. Diversion and closure of the river was completed in the Stage VI contract in November 1979. By December 1981, all of the generators had been installed but testing had not been completed. By February 1982 units 4, 5 and 6 were synchronized to the power line and were operating and mechanical problems with the stub shaff bearings had been resolved. All units are expected to be operating by August 1982 and pump-back testing is expected to be completed by April 1983.

1-05. Contracts:

a. Stage I: Contract ENG-41-66-228 was awarded to Clarkson Construction Co., Kansas City, Missouri, on 10 September 1966, for a low bid of $933,575 and was completed 10 January 1967. The following items of work were involved:

(1) Excavate cutoff trench to bedrock on the right abutment from Sta. 22+50 to Sta. 38+00 and in the valley section from Sta. 48+75 to Sta. 63+25 and install a single-line grout curtain along the dam axis and backfill trenches.

(2) Excavate overburden in the Spillway-Powerhouse area to bedrock (El. 630 ±) and conduct experimental blasting of bedrock.

(3) Excavate the outlet channel to bedrock from Sta. 55+00 to Sta. 65+00 (control line "A").

(4) Construct temporary dikes, to El. 675, around the embankment excavation area and install well point system to bedrock to control ground water seepage thru protective dike parallel to Osage River.

(5) Place embankment materials to El. 667 and rockfill in protective dikes to El. 660.

b. Stage II: Contract DACW-41-68-C-0006 was awarded to Western Contracting Corp., Sioux City, Iowa, 21 July 1967 for a low bid of $2,294,450 and was completed 18 October 1969. The following items of work were involved:

(1) Excavate overburden in approach channel to El. 660 from Sta. 30+00 to Sta. 42+00 (control line "A").

(2) Construct protective dike in outlet channel at Sta. 72+00 (Control Line "C") and excavate overburden in outlet channel to El. 630 downstream of protective dike to Sta. 90+00 (control line "C").

(3) Continue bedrock excavation in Spillway-Powerhouse area to El. 616.

(4) Excavate overburden between outlet channel dikes to El. 630 from Sta. 62+00 to Sta. 72+00 (control line "C").

(5) Place embankment materials to El. 727 from Sta. 50+00 to Sta. 61+00 (dam axis) and construct rim dikes.

(6) Excavate rockfill in the upstream berm from dam axis Sta. 55+15 to Sta. 57+00 and place in rock Zone 1.

(7) Place slope protection on upstream face of embankment and on side slopes of outlet channel from Sta. 62+00 (control line "C") to the downstream protective dike.

(8) Place rockfill on right bank of river at range 5+38 upstream and place riverfill on left bank at range 4+96 downstream.

c. Stage III: Contract DACW-41-71-0043 was awarded to Guy F. Atkinson Co., San Francisco, California, 30 October 1970, for a low bid of $36,294,652 and was completed 24 September 1974. The following items of work were involved:

(1) Construct upstream approach channel plug to El. 675.

(2) Complete embankment to El. 755 from dam axis Sta. 22+00 to Sta. 38+51 and from Sta. 48+00 to STa. 58+00 and construct tie-ins to bulkheads.

(3) Complete final rock excavation of Spillway-Powerhouse, stilling basin, intake channel upstream of concrete to the IV on 5H slope at Sta. 44+75 (control line "C").

(4) Construct concrete bulkheads, powerhouse, stilling basin and spillway crest to El. 660.

(5) Construct grout curtain and drain holes under Spillway-Powerhouse from inside grouting gallaries.

(6) Operate Government Quarry 3 for production of coarse and fine aggregate for concrete, riprap, bedding spalls and transition material.

d. Stage IV, Sterrett Creek Dike: Contract DACW-41-74-C-0231 was awarded to List and Clark Construction Co., Overland Park, Kansas, 28 May 1974 for a low bid of $3,855,867 and was completed 3 May 1976. The following items of work were involved:

(1) Excavate cutoff trench to bedrock on right abutment from Sta.9+70 to Sta. 11+38.

(2) Install a single line grout curtain between the above stations.

(3) Excavate inspection trench from Sta. 11+38 to Sta. 85+60.

(4) Backfill trenches and place embankment to El. 757.

e. Stage V: Contract DACW-41-75-C-0015 was awarded to Truman Powerhouse Contractors, a joint venture composed of Guy F. Atkinson Co. and Wismer and Becker Contracting Engineers of San Francisco, California, on 8 August 1974 for a low bid of $29,266,294 and was completed 6 February 1981. This work involved installation of electrical generating equipment in the powerhouse.

f. Stage VI: Contract DACW41-77-C-0049 was awarded to S.J. Groves and Sons Co., Springfield, Illinois, January 1977 for a low bid of $13,181,628 and was completed November 1979. The following items of work were involved:

(1) Complete the embankment tie-ins around the concrete structures.

(2) Remove approach and outlet channel plugs.

(3) Construct rock dikes across Osage River and divert river flow through spillway.

(4) Install dewatering system, remove water and muck from closure area.

(5) Install 3-line grout curtain from dam axis Sta. 63+36, up left abutment to Sta. 74+81.

(6) Construct embankment across Osage River channel and tie-in to left abutment.

(7) Construct roadway across top of dam and over Spillway-Powerhouse.

(8) Construct hoisting equipment over powerhouse.

(9) Extend concrete spillway wier to final elevation, using temporary bulkheads.

1-06. Contract Supervision: The project was constructed under the supervision of the Harry S. Truman Project Office, US Army Corps of Engineer District, Kansas City, Missouri. Resident Engineers were Mr. Richard F. Griffith, Sep. 1966 to Dec. 1968; Mr. Walter E. Donan, Dec. 1977 to Dec. 1978; Mr. D. J. Bell, Dec. 1978 to Mar. 1979; and Mr. Billy J. Cheatham, Mar. 1979 to April 1983. Project Geologists were Mr. John W. Doty, Sep. 1966 to Mar. 1978 and Mr. Wallace E. Penn, Mar 1978 to June 1980. The Grouting Subcontractor for Stages I and IV was the Judy Drilling Company of Kansas City, Kansas. Grouting for Stage III was performed by Minnesota Services Co., Pasadena, California and grouting for Stage VI was performed by Continental Drilling Company, Seattle, Washington.

CHAPTER 2

FOUNDATION EXPLORATION

2-01. Investigations Prior to Construction consisted of field reconnaissance, study of aerial photos, review of published literature and drilling and sampling of overburden and bedrock. A total of 338 borings and test pits were completed by Government drill crews prior to Stage I construction. A summary of drilling is shown in Table 1.

Table 1

Borings Drilled Prior to Construction

Type	Number
Core holes	
36-inch calyx	4
6-inch	20
NX	79
Drive holes	133
Drive & NX core	41
Drive & 6-inch core	8
Undisturbed push	10
Undisturbed & NX core	13
Undisturbed & 6-inch core	4
Undistrubed & drive	6
Vane shear test holes	11
Auger	1
Test pits	9
Total	338

Drive holes were made with 6-inch, 4-inch and 2-inch diameter drive barrels. Undisturbed soil samples were taken with 5-inch and 2-inch diameter Shelby tubes. Five, NX angled core holes, were drilled in the left abutment and 10 NX angled core holes were drilled in the embankment area. Fourteen angled NX core holes were drilled in the Spillway-Powerhouse area. Fourty-eight bedrock borings were pressure tested with water. Twenty-eight borings "took" from four to seven gallons per minute and four borings "took" over seven gallons per minute. The higher "takes" were largely in the upper weathered zone of the bedrock. Water "takes" in the breccia zones were no greater than in other rock. See Plates 36 and 37.

2-02. Investigations During Construction: During construction Government drill crews completed 140 additional borings and the Contractor drilled 123*, three inch diameter Air-Trac holes to grout solution cavities in the powerhouse foundation and 50 Air-Trac holes to grout solution cavities in the right abutment cutoff trench. A summary of drilling is shown in Table 2.

Table 2

Borings Drilled During Construction

Core holes		
6-inch	4	
NX	79	
Drive holes	36	
Drive & NX	2	
Auger	7	
Test pits	48	
Government Drilled Total	140	
Air-Trac		
3-inch grout holes	123*	Powerhouse foundation
3-inch grout holes	50	Cutoff trench
Contract Drilled Total	173	

*Note: See paragraphs 10-1 (a) and (b).

CHAPTER 3

GEOLOGY

3-01. Regional Geology: Harry S. Truman Dam and Reservoir is located in the northern one third of the Osage Plains section of the central lowland Physiographic Province, a region characterized by old scarped plains beveling faintly inclined strata. The main streams flow in incised meandering channels. The valleys are comparatively wide and are commonly cut 100 feet below the adjacent rolling upland. Along its eastern border, the area is underlain by a narrow outcrop band of thick Mississippian limestones, interbedded with a few thin shales and sandstones. West of the Mississippian outcrop, the area is underlain chiefly by Pennsylvanian shales and thinner beds of limestone, sandstone and coal. Differential weathering of the westward dipping Pennsylvanian strata has produced broad, gently rolling surfaces on the shales, alternating with narrower and higher eastward-facing escarpments developed on the outcropping limestones.

3-02. Site Geology: At the damsite, Osage River valley is about 5,000 feet wide, and is cut about 200 feet below the adjacent upland. Steep to vertical bluffs, broken by short, deep, ravines form the left valley wall both upstream and downstream of the dam axis. The river, which flows along the toe of the left abutment, is bordered on the right by a flood plain about 1,500 feet wide at approximate El. 660 m.s.l. The right side of the valley rises rather abruptly from the flood plain to the top of an alluvial terrace at El. 685 m.s.l., flattens out for a short distance and continues to rise gently to El. 800. South Grand River and Tebo Creek, which presently enter Osage River about one mile upstream of the dam, once flowed east thru Sterett Creek valley and entered Osage River about four miles downstream of the dam. Lateral erosion by Osage River caused capture of this drainage, bringing it to the west, and abandonment of its former channel. Sterrett Creek dike impounds this arm of the reservoir. See Plate 1.

3-03. Description of Overburden:

a. Valley Alluvium: Valley alluvium at the damsite is 30 to 45 feet thick and consists of a basal layer of sand and gravel 5 to 15 feet thick, ovrlain by 20 to 30 feet of stiff to hard lean clay. The lower part of the left abutment, below the steep Burlington limestone bluff, has a cover of 10 to 25 feet of talus composed of clay and boulders. See Plate 3. The character of the overburden along the axis of the Sterett Creek dike is shown on Plates 88, 89 and 90. The thickest overburden, about 75 feet, is located on the right side of the valley. It consists of a basal gravel layer, 10 feet thick, overlain by 65 feet of lean and fat clay. Outside the main channel, valley alluvium ranges from 7 feet of clay capping a bedrock high in the middle of the valley just upstream of the dike, to 30 to 40 feet of clay blanket overlying 20 to 30 feet of generally well graded sand and gravel.

b. Terrace Deposits: The right abutment has a cover about 20 feet thick of residual clay and gravelly clay and a remnant of a former alluvial terrace deposit of clayey, sandy gravel between El. 670 and 690. The terrace is recognizable at the dam axis but upstream and downstream it has largely been destroyed by erosion.

c. Upland Overburden averages less than 10 feet thick and is composed of residual clay with a large amount of chert gravel and lesser amounts of dolomite limestone and sandstone fragments.

3-04. Bedrock Stratigraphy: At the damsite and over a considerable area of Missouri no Silurian or Devonian age rocks have been recognized and erosion has removed all rock, (if originally present) deposited after Jefferson City time and prior to Mississippian time. The blank in the rock column is estimated at 80 million years. Subsequent to Mississippian time erosion has removed all rock above the Burlington limestone. Burlington and the underlying Pierson, Northview and Sedalia-Compton formations were deposited on the eroded surface of Ordovician rocks (Cotter-Jefferson City dolomite limestone). At the present time the only outcrops of Burlington limestone in the area are outliers, capping hills of less resistant Ordovician strata. The remaining Burlington limestone was originally deposited in sink holes on the ancient Ordovician rock surface, (left abutment and Quarry Sites 3 and 4.). Bedrock strata exposed at the damsite belong to the lower Mississippian, Devonian-Mississippian and lower Ordovician Systems. The units are shown on the geologic column on Plate 4 and Geologic Map on Plates 6 and 7 and are described in descending order in the following paragraphs.

a. Mississippian System: Rocks of the Mississippian System are exposed in northeastern, central and southwestern parts of Missouri and occupy about one fourth of the State's total surface area. In the central part of the State, these rocks total about 400 feet in thickness. The uppermost bedrock unit present at the damsite is the Burlington Formation which is in the lower part of the Mississippian System.

(1) Burlington Formation is a white to light buff, moderately hard, medium bedded to massive coarsely crystalline, fossiliferous, crinoidal, limestone. Layers of white to gray, chert nodules, are common, especially in the upper part. In central Missouri, thickness ranges from 75 to 100 feet, at the damsite, its thickness is about 60 feet. Kaysinger Bluff (left abutment) is capped by Burlington limestone.

(2) Pierson Formation is moderately hard, massive, finely crystalline, dolomitic limestone about 4.5 feet thick. The contact with overlying Burlington and underlying Northview is conformable. (i.e. no disruption of deposition).

(3) Northview Formation is gray to greenish gray, soft to moderately hard, silty, massive, laminated to blocky shale about 1.5 feet thick. Contact with the underlying Sedalia-Compton Formation is conformable.

(4) Sedalia-Compton Formations (Undifferentiated): These two formation have been classified together as the Chouteau Group. The unit is made up of thin to thick beds of light gray to medium gray, moderately hard, finely crystalline, aggrillaceous and dolomitic limestone. Occasional shale bands, partings, laminae and light and dark gray chert nodules occur throughout the unit. Thickness is about 35 feet. Contact with the underlying Sylamore sandstone is unconformable.

b. Devonian-Mississippian System:

Sylamore (Bushberg) sandstone: The precise age of this bedrock unit has not been established but the lithology of the rock and stratigraphic position suggests the age to be late Devonian or early Mississippian. At the damsite the unit is a single massive bed of conglomertic sandstone about one foot thick. It is argillaceous to shaly, calcareous, soft to moderately hard, fine grained, well cemented, light brown to green and contains shale laminae.

c. Ordovician System:

Cotter-Jefferson City Formations (undifferentiated): Rocks of the Ordovician System are exposed over approximately one-third of Missouri and attain an aggregate thickness at about 3,800 feet. At the damsite only the lower one-half is present. After deposition of the Cotter and Jefferson City formations and before deposition of the overlying Sylamore sandstone, approximately the upper one-half of the Ordovician Strata, all of the Silurian, and some, if not all of the Devonian Strata, a combined thickness of possibly 1,800 feet, were removed by erosion. At the damsite, the two remaining formations total about 300 feet in thickness. They have not been differentiated but since the formations form the foundation for the Spillway-Powerhouse structure and extensive excavation, (75 ± feet) was required, the upper 230 feet at the damsite have been subdivided into 24 identifiable units. See Plate 4. The Cotter-Jefferson City is a moderately hard dolomite limestone that gives the overall impression of being thick-bedded to massive. The texture ranges from dense to sugary. The rock is also frequently solution pitted to vuggy with some of the vugs filled with tripolitic chert. The rock ranges from gray to light gray and weathers to brownish gray. Chert in nodules, lenses or beds and sandstone beds, less than 6 inches thick, are scattered throughout the formation. Beds of laminated dolomite and green shale up to 4 feet thick are common in the upper 100 feet of the formation. Below unit 11, the shale occurs only as partings and very thin beds except at the 18-19 and the 23-24 unit contacts. Much of the formation contains intraformational breccia as distinct from solution and fault breccia of more recent origin. Cotter-Jefferson City is underlain by similar dolomite and sandstone strata of the Ordovician and Cambrian Systems to a depth of 1,000 feet or more.

3-05. Bedrock Structure: Geologic history of the Ozark area is fairly complex. Dolomite-limestones, limestones, shales and sandstones of the Paleozoic era were deposited and later removed under varying conditions. Several cycles of Tectonic disturbances, ranging from uplifts of the Ozark dome to slight tilting, peneplanation and deep solutioning have occurred. The

ologic history from the deposition of the Cotter-Jefferson City formation to e present, spans a time period of almost 400 million years. A certain ount of masking of earlier features has taken place due to later geologic ppenings. The damsite is located on the northwest edge of a large liptical dome, the axis of which, trends northeast-southwest, from the St. ancois Mountains in Iron County, Missouri to the vicinity of Springfield, in een County, Missouri. Regional dip is 10 to 20 feet per mile to the rthwest. Bedrock topography and structure at the damsite is shown on ate 5. Structure contours were drawn on the contact of units 18-19. No jor faults were found in the excavations and no evidence has been found that drock structure controls the location of major streams in the area. It is obable that some of the bedrock structure, including a considerable portion the breccia, found in west-central Missouri and at the damsite, is related solutioning of Ordovician dolomite limestones and Mississippian limestones. eccia zones were encountered in the abutments and right bank terrace area it they were of limited areal and stratigraphic extent and are considered to interformational, i.e. the result of limited partial collapse which ccurred contemporaneous with deposition. They caused no major problems iring construction.

3-06. <u>Earthquake History:</u> The reservoir is located in an area elatively free from recorded earthquake activity. The nearest area of oderate to severe activity is about 230 miles at New Madrid, Missouri. The ew Madrid quakes of 1811 and 1812 had an intensity of X on the modified ercalli scale. The damsite is in Zone 1 on the Seismic Probability Map of he United States. The Spillway-Powerhouse and nonoverflow structures were esigned with a horizontal earthquake acceleration of 0.05 g.

3-07. <u>Bedrock Weathering:</u> Weathering effects on Burlington limestone in he left abutment consist of softening, clay filling, widening and staining of ertical joints and fractures. These effects extend only a few inches from he joint or fracture faces. Weathering effects on the Cotter-Jefferson City olomite limestone consist of widening, staining and softening of the rock in he vicinity of joints, fractures and bedding planes. Weathering effects xtend to a depth of 2 to 6 feet in the valley and up to 20 feet in the butments.

3-08. <u>Leaching or Solution Activity:</u> Solution cavities up to several eet across were encountered during excavation of the cutoff trench from ta. 22+90 to Sta. 23+50. See Plates 13 and 14 and Photos 26 thru 32. These avities were dugout and backfilled with concrete. Smaller solution channels ere "broken out" and grouted. See Plate 116. Small solution channels in the pillway-Powerhouse foundation were also "broken out" and grouted. See Plate 4 and paragraph 10-1. No serious problems resulted from solution cavities.

3-09. <u>Ground Water:</u> Ground water at the damsite is at approximately l. 670. Water wells in the area, which penetrate the Gunter sandstone, at a epth of about 500 feet and deeper formations, yield 40 to 200 gallons per inute of potable water. On the right bank terrace, bedrock is covered with bout 40 feet of alluvial fill. The lower 10 to 20 feet is a poorly graded layey sand and gravel overlain by generally stiff lean clay. In the river

channel bedrock is covered by 8 feet of poorly graded, clayey, sandy gravel which is overlain by 2 to 8 feet of lean clay and fat clay muck. During excavation of the cutoff trench, a small amount of ground water entered the excavation from the basal gravel layer. Sandbag dikes, sumps and gasoline powered pumps were used to control and remove the water. During closure, perforated collector pipes were placed in the basal gravel and ground water entering them was pumped out. See paragraph 5-02, Plates 103 and 104 and Photos 191, 192, 193 and 194. Ground water was not a serious problem during construction.

3-10. <u>Engineering Characteristics of Overburden Materials:</u> Laboratory tests on samples of the alluvial clay indicated the material would provide a suitable foundation for the embankment. Design shear strength in the "Q" test was: Tan Ø = 0 and cohesion = 0.75 tons per foot. For the "R" test, Tan Ø = 0.2 and cohesion = 0.70 tons. For the "S" test, Tan Ø = 0.4 and cohesion = 0. In unconfined compression, design shear strength was, cohesion = 0.75 tons per foot. For compacted impervious embankment clay design strength in the "Q" test was Tan Ø = 0.15 and cohesion = 0.6 ton per foot. In the "S" test Tan Ø = 0 and cohesion = 0.6 ton per foot. For more complete test data see Design Memo No. 13., Soil Data and Embankment Design

3-11. <u>Engineering Characteristics of Bedrock Materials:</u> Cotter-Jefferson City Dolomite Limestone Formation is the foundation for the Spillway-Powerhouse structure. Total thickness of the formation is about 300 feet. The structure is founded about 75 feet below the top of rock at El.589 ± in Zones 17 and 18. See Plates 49 thru 59. Immediately below Cotter-Jefferson City are about 100 feet of Roubidoux sandstone, dolomite sandstone and dolomite limestone. Cotter-Jefferson City formation is chiefly hard, thin to medium bedded, cherty, crystalline to dense textured dolomite limestone. Bedding planes are irregular but tight. Individual beds range from a few inches to about 3 feet thick. Shaly dolomite occurs as thin, mostly continuous, irregular partings. Intraformational breccia occurs in limited irregular zones and consists of fragments of chert and dolomite in a matrix of dolomite. Shale breccia occurs in relatively undisturbed bedrock filling joints, fractures, and pockets and along bedding planes. A well defined system of joints is not apparent. The most significant features of the rock are the thin irregular beds and the numerous tight, closely spaced fractures. These fractures, mostly at right angles, and nearly vertical, occur in individual beds but rarely extend beyond bedding planes. Most of the fractures are healed or filled with shale or shale breccia. Open fractures are not extensive. Unconfined compressive strength ranges from 1,300 to 9,500 p.s.i. and averages 6,100 p.s.i. Modulus of Elasticity ranges from 0.17×10^6 to 2.13×10^6 and averages 1.34×10^6. Dry density averages 153 pounds per cubic foot. Triaxial and direct shear tests were performed on selected samples. Based on these tests, design shear srength, normal to bedding was tan Ø = 0.65 and cohesion = 50.4 k.s.f. Traceable dolomitic shale partings and bands and stylolitic partings, within correlated units, were classified into zones. These zones, "A" thru "P" are shown on the Geologic column and Legend and on Geologic Profiles. See Plates 4 and Plates 36 thru 38. Tests were performed on samples oriented so the weak planes were

CHAPTER 7

TUNNELS, SHAFTS AND UNDERGROUND STRUCTURES

7-01. Tunnels and Shafts: Tunnels, shafts and underground structures were not constructed.

CHAPTER 6

PILE DRIVING OR CONSTRUCTION OF OTHER SPECIAL FOUNDATIONS

6-01. Slurry Wall Cutoff: As an alternate to the excavation of the toe trenches in the river channel required by the Stage VI Contract during closure (Section 2B, paragraph 11), the Contractor submited a value engineering proposal to install a dewatering system. The proposal was adopted. It consisted of a slurry wall bounding the river dikes, both upstream and downstream, and connecting into a deep well dewatering system alined normal to to the dam axis in the vicinity of Sta. 61+00. See Plates 103 and 104 and paragraph 5-02. Slurry Systems Specialists Inc., East Chicago, Indiana was contracted to install the slurry wall. A special machine mounted on a Model 3900 Manitowac Crane was used. It consists of a larger vibrator and beam which vibrates vertically through the overburden to firm bedrock. The beam which measures 33 inches in webb and 3 inches thick injects a bentonite slurry through two orifices as the beam advances into the ground. This serves as a lubricant for the beam and assures that the orifices remain open. As the beam is withdrawn, slurry is injected into the void producing a wall. The wall measures 3 1/2 inches thick. Flanges on the beam assist in alinement. A 14 inch fin serves to allow for a minimum overlap of 6 inches to assure continuity. The beam is advanced to the point of refusal, withdrawn and readvanced, at a rate of about 3 meters per minute, to form a continuous curtain through the overburden or embankment to refusal on bedrock. Often a shallow trench is dug in the ground surface with a backhoe along the trace of the wall to provide a slurry reservoir. The slurry mix consisted of: 200 lbs. bentonite, 384 lbs. cement, 20 lbs. TriCalcium Phosphate and 54 cu. ft. of water. While advancing the wall across the closure area both the upstream rock fill and the downstream rock fill displayed unstability by slides, cracking and settlement. Stiffer mixes were used when these conditions were encountered. The slurry wall was effective in reducing the amount of ground water entering the area during closure.

inch mesh, chain-ling fence was ued on all excavated rock surfaces higher than 15 feet and on slopes 1V on 0.5H or steeper. The fencing was secured by rock bolts. Vertical bolts, 5 feet long were installed, three feet back from final surfaces prior to presplitting. All excavating and hauling equipment was equipped with rollbars and backup warning signals. Wood stairs were provided on the steep left abutment during grouting. Five minute warning blasts were sounded and men and machines were not permitted within 500 feet before explosive blasts were fired.

1.5 feet of hole. Results were generally good except for damage to the upstream corner of the spillway wall which required repair with rock bolts and concrete. See Plate 74 and Photo 53.

5-09. Foundation Preparation:

a. Embankment Section: Prior to placement of fill in stripped areas, standing water was drained from the foundation surface and the sides of ponds, stump holes, test pits, and other similar depressions were broken down. Depressions were filled with material appropriate for that zone of the embankment and graded to provide a generally level surface. Existing dug wells were backfilled with impervious clay, procesed and compacted as required. Existing drilled wells were grouted. Earth foundation areas to receive fill were stripped and thoroughly loosened by plowing or disking to a depth of 6 inches and compacted with a minimum of 6 passes of a tamping roller.

b. Cutoff Trench: A cutoff trench was excavated to bed rock, along the dam axis, beginning on the right abutment at Sta. 22+30 and extending to Sta. 38+20, at the right nonoverflow monolith. The cutoff trench continues at the left nonoverflow monolith at Sta. 48+30 and extends across the valley and up the left abutment to Sta. 70+50. The side slopes were cut 1V on 2H. The bottom width was 43 feet for most of the trench but it was widened to 135 feet in the closure area. For details see Plates 40 thru 43. For map of bedrock in the cutoff trench, see Plates 13 thru 34. Solution cavities and clay filled caves were encountered from Sta. 23+00 to Sta. 25+50. The clay fillings were dug out with shovels and the cavities were backfilled with concrete. See Photos 26, 29, 30, and 31. Smaller clay pockets and cavities were broken out and grouted shut. Grouting was performed on bedrock on the floor of the cutoff trench. Ground water entering the cutoff trench was collected with sandbag dikes and sumps and pumped out with gasoline powered pumps. The rock surfaces were cleaned with shovels, brooms and air-water jets. Open joints were cleaned out and backfilled with impervious clay.

c. Left Abutment: Overburden on the left abutment was stripped off for a distance of 250 feet upstream and 300 feet downstream. The abutment was initially stripped of overburden in 1964 and was allowed to weather until 1978. Shale breccia extends from Sta. 70+20 at El. 622 to Sta. 72+00 at El. 729 which is the base of the Sedalia-Compton limestone. The upper one to two feet of the breccia was stripped off immediately before placing impervious fill in the central core section of the embankment. Downstream of the impervious core the breccia was stripped to solid material and 5 feet of pervious was placed against it. On the upstream side the breccia was stripped to solid material and a lean clay blanket 5 feet thick was placed on top of breccia. The protective clay blanket extends upstream for 650 feet. See Plates 92 and 93.

5-10. Safety Precautions: A 42 inch high chain link fence was constructed around Excavation Areas A and B. Bedrock surfaces were scaled immediately after being exposed to prevent detached, loose, and overhanging rock from creating a safety hazard. Draped fencing with 9 gage, galvanized, 2

50 grain primacord downline and fired with a single blasting cap. Results were good. A solid column of Trimtex in holes spaced 3 feet and 4 feet apart was used but produced more radial fractures in the drill hole casts and in the presplit face."

b. The Primary Objective of the test blasting program was to develop an efficient method of rock excavation that would produce a minimum amount of minus 2 inch size material and a maximum amount of minus 3,000 pound stone. Excavation of Cotter-Jefferson City dolomite-limestone at the Stockton Dam Project resulted in an excessive amount of minus 2 inch size material (27 to 53 percent). Some of the minus 2 inch material was the result of careless handling of the blasted rock and some was believed to be the result of inadequate blasting technique. Unconfined compressive strength averaged 5,800 p.s.i. Dry density averaged 156 lb./ft.3. Siesmic velocity was not measured but blasting consultants recommended use of low density, low velocity explosives (max. 5,000 ft./sec.). At Truman damsite, unconfined compression averaged 6,100 p.s.i. and dry density averaged 153 lb./ft.3. Siesmic velocity measured 7,000 ft./sec. In the test blasting area 31 percent of the beds are less than 6 inches thick, 10 percent are less than 2 inches thick and 48 percent are 2 feet thick or more. The rock also contains numerous high angle fractures in two directions. They are spaced from 1/4 inch to 2 inches apart but do not cross the bedding planes. Some are open and some are partially healed. These minute fractures are believed to be responsible for the large amount of minus 2 inch size material (20.7 to 31.6 percent, average 31 percent).

c. Procedures: From experience gained in the Stage I test blasting, the following procedures were specified for the Stage II and Stage III Contracts: (1) Distance between blast holes shall be twice the distance between rows (to control maximum size of rock particles). (2) Blast holes shall be 2 3/4 inch diameter and explosives shall be 1 1/4 inch diameter, low velocity cartridges (max. 7,500 ft./sec.). (3) Powder factor shall not exceed 1/2 pound per cubic yard. (4) Subdrilling shall not exceed 2 feet and bench height shall not exceed 20 feet. (5) All shots shall be fired with delays to an open face with all muck from previous shot removed. About 405 presplit and excavation shots were made in the Spillway-Powerhouse excavation. For more details see Supplements A, B, C, and D.

5-08. Line Drilling and Presplit Blasting: Specifications required that all final rock surfaces with a slope of 1V on 1H or steeper shall be excavated by presplit blasting. Each presplit hole could not deviate more than 6 inches from the presplit surface at the bottom of the hole. For faces, 15 feet or deeper, presplit holes were drilled no more than 3 feet apart. For faces, less than 15 feet, the maximum spacing was 18 inches. All presplit holes were 2 3/4 inch diameter and explosives were 7/8 inch diameter cartridges with a maximum velocity of 12,000 ft./sec. and bulk strength of not more than 40 percent. Holes were spaced 3 feet apart and were loaded with one 7/8 inch by 24 inch cartridge per 3 feet of hole. External corners were chamfered 6 feet back along each wall with 5 equally spaced intermediate presplit holes. The intermediate holes were loaded with one 7/8 inch x 4 inch cartridge per

c. Upstream Rock Slope Protection, 65,612 c.y. was selectively loaded, more durable rock, excavated from Area A. The material was reasonably well graded and limited to a maximum weight of 1,000 pounds with 30 percent larger than 400 pounds and 5 to 15 percent less than 10 pounds. It was placed in a 2 foot layer on the upsteam face of the dam, from natural ground to El. 717 on right side of the Spillway-Powerhouse and from natural ground to El. 703.5 on the left.

d. Downstream Rock Slope Surfacing, 84,960 c.y. was selectively loaded, more durable rock, excavated from Area A. The quality of the rock used was determined by the Contracting Officer. It was placed in a 2 foot layer on the downstream face of the dam.

e. Channel Scalped and Channel Choker Course, 40,832 c.y: Channel scalped rock was produced from required excavation in Area A. It was well graded stone, all minus 18 inch with a maximum of 10 percent minus 2 inch. An 18 inch layer was placed on both sides of the outlet channel from the downstream channel plug to the Osage River. Channel Choker Course was produced from required rock excavation in Area A. It was all minus 10 inch with 70 to 95 percent passing 4 inch and 45 to 70 percent passing 2 inch. Not more than 15 percent of the material passing 2 inch was finer than No. 200 sieve (washed). A 12 inch layer of Channel Choker Course underlies the channel scalped rock in the outlet channel.

5-06. Rock Tests: Gradation and density tests were performed on rock materials from required excavation. Density of compacted Zone 1 and Zone 2 rock ranged from 118.7 lbs. per cu. ft. to 158 lbs. per cu. ft. and averaged 131.6 lbs. per cu. ft. in eight tests. Dry density of the rock is 153 lbs. per cu. ft. These figures indicate about 16 percent voids in the rock mass which is acceptable in a rock fill. For more details see Supplements A and B.

5-07. Blasting:

a. During Initial Rock Excavation in the Stage I Contract a test blasting program was conducted, in a limited area of the powerhouse excavation, under the direction of Mr. Robert Stansfield, Geologist and Blasting Specialist with the Kansas City District, Corps of Engineers. His report on the test blasting is on file in the KC District Office. The following information is taken from Mr. Stanfield's Report. "From 21 October to 26 November 1966, 28 test blasts were fired and 25 field gradation tests were performed on samples of the broken rock. The blasts were designed to be both experimental and productive as the rock was used in construction of the embankment. Bench heights of 15 and 20 feet were used. Most of the blast holes were 2 3/4 inch diameter. They were vertical and overdrilled two feet on a diamond pattern. The space between holes was generally twice the distance between rows. Stemming was 4 to 5 feet and powder factor ranged from 0.31 to 0.59 lbs./c.y. Blasts holes were loaded with 1 1/4 inch diameter cartridges and fired with delay blasting caps at the bottom of the hole. A few blasts were fired with blasting caps and primacord. Final faces were excavated 1V on 0.75H by presplit blasting. Presplit holes were drilled on 2 feet centers, loaded with 6 inch lengths of Trimtex spaced 2 feet apart on

d. During the Stage VI Contract, rock excavation totalled approximately 54,000 cubic yards and was primarily performed: (1) in the cutoff trench in the river channel area and left abutment, (2) as directed in localized areas up to 550 feet upstream of the dam axis on the left abutment below the Sedalia-Compton limestone, (3) as directed in the Spillway-Powerhouse area and (4) as directed in the embankment foundation. Foundation excavation-rock was divided into two classes: Class I rock was defined as removal of materials consisting of limestone and dolomite-limestone, as shown on the drawings and boulders or fragments of dolomite-limestone greater than 1 cubic yard in volume. Class I rock was removed by ripping, presplitting, light blasting and handwork. All final slopes in Class I rock were presplit. Class II rock was defined as removal of materials consisting of shale, shale breccia and sandstone breccia as shown on the drawings. Shale breccia and sandstone breccia consisted of chert and limestone pebbles and fragments in a shaly or sandy matrix and were very weather-sensitive when exposed to the elements. Blasting of Class II material was not permitted. Temporary and permanent slopes in shale and sandstone breccia were not steeper than 1V on 1H and the average slope of any type of rock face was not steeper than 1V on 1H. The excavated surfaces were protected at all times so that the shale breccia and sandstone breccia received minimal exposure to freezing, wetting, or drying conditions. Class II rock excavation was coordinated with placement of dam embankment fill material so that the final foundation surfaces did not remain exposed longer than 24 hours before being protected with compacted embankment material.

5-05. Rock Products: The following rock products were produced from required excavation of the Cotter-Jefferson City Dolomite-Limestone. See Table 5 and also Supplement A.

a. Scalped Rock and Choker Course 191,685 c.y. Scalped Rock was all minus 18 inch with not more than 10 percent minus 2 inch. It was placed in an 18 inch layer forming a part of the horizontal drain downstream of the pervious zone, above Zone 2 rock, and as a part of the tie-ins with the concrete nonoverflow sections. See Plates 41 thru 43. Choker course was also used behind the tailrace training wall above top of rock, around the cable tunnel, downstream of the gravity wall and in the parking area on the spillway side of the concrete structure.

b. Zone 1 and Zone 2, Rock 1,323,533 c.y.: Zone 1 material was quarry run rock from excavation in Area B. It was mostly smaller than 18 inch and generally contained about 20 percent plus 6 inch and about 35 percent minus 2 inch. It was placed on the upstream side of the clay core and the larger pieces were pushed to the outer upstream 20 feet with a rock rake. Zone 2 rock was selectively loaded, more durable rock, excavated from Area A. It was all minus 36 inch size. It was used for the 9 foot thick horizontal drain on the downstream side of the core. It was also used in a mass section on the upstream side from El. 740 to the crest. Zone 2 rock was also placed adjacent to Zone 3 rock in the nonoverflow tie-ins.

Table 5
MATERIAL DISTRIBUTION STAGE III

Major Embankment Pay Items	Approximate Quantity (c.y.) to be Placed in Dam	Approximate Quantity (c.y.) to be Placed in Permanent Stockpiles	Source
Impervious	227,000		Overburden excavation or borrow
Random	242,000		Overburden excavation or borrow
Berm	153,000		Overburden excavation or borrow
Area Fill	495,000		Overburden excavation or borrow
Pervious	21,000		Offsite
Transition	7,000	6,000	Quarry
Scalped rock zone	20,000	21,000	Overburden excavation or borrow
Channel scalped rock	5,000		Spillway-powerhouse rock excavation
Choker course	57,000	54,000	Spillway-powerhouse rock excavation
Channel choker course	3,000		Spillway-powerhouse rock excavation
Rock zone 1	283,000	*	Spillway-powerhouse rock excavation
Rock zone 2	216,000	472,000	Spillway-powerhouse rock excavation
Rock zone 3	12,000		Quarry
Rock slope protection	17,000	30,000	Spillway-powerhouse rock excavation
Rock slope surfacing	30,000	40,000	Spillway-powerhouse rock excavation
Riprap (including types A and B)	46,000		Quarry
Spalls	14,000		Quarry
Bedding	7,000		Quarry

*Rock materials remaining after completion of embankment and not suitable or required for other permanent stockpiles.

a. During the Stage I Contract rock excavation was limited to about 5,000 cubic yards of unsatisfactory bedrock in the cutoff trench and about 40,000 cubic yards in the outlet works structure area. The purpose of the initial rock excavation in the outlet works was to determine the effectiveness of presplitting techniques, to expose sound rock walls for geologic study and to determine the fragmentation characteristics of the rock using various blasting techniques in order to maximize usage of the excavated rock in the dam.

b. During the Stage II Contract, rock excavation totalled approximately 504,967 cubic yards as shown in Table 4 below.

Table 4

Rock Excavation Quantities Stage II

Area	Cubic Yards
Spillway-Powerhouse (Area A)	259,516
Embankment Foundation	3,000
Approach & Outlet Channels (Area B)	242,451

Tolerance for bottom surface of excavation in area "B" was 1 foot plus to 3 feet minus and for slopes, plus or minus 2 feet. For area "A" tolerance for the bottom surface and slopes was plus or minus 3 feet. Using experience gained in rock excavation in the Stage I contract, the Gove nment furnished shot designs to the Contractor for the early shots. All blasting was performed to produce the maximum amount of well graded material and minimize the quantity of material finer than 2-inch. During the Stage II and Stage III contracts, which involved the bulk of rock excavation, blasting records of each shot were maintained and methods were modified to obtain the most desirable fragmentation for each rock product. See Supplement D. Initial drilling and blasting in any area or rock type was limited to 2,000 cubic yards per shot until it was proven the method would produce the desired results. Subsequent drilling and blasting was limited to 10,000 cubic yards per shot in the approach channel and outlet channel. Excavated rock was placed in the appropriate zone of the embankment or stockpiled as directed. Special processing was performed as necessary. Poor quality rock, as designated by the Contracting Officer, was placed in rock Zone 1.

c. Stage III Rock Excavation was chiefly for the Spillway-Powerhouse and totalled about 1,183,000 cubic yards. Table 5 next page below shows the material distribution of rock excavation during Stage III.

water levels during dewatering. Under seepage was further controlled by pumping out water from 6-inch diameter perforated steel pipe placed in the basal gravel zone about 10 feet above bedrock. There were no unusual or unforseen problems with ground water or river water. See Plates 103 and 104.

5-03. Overburden Excavation was defined as the removal of materials between top of ground and top of bedrock as shown on the drawings, and included topsoil clay, silt, sand, gravel or combinations of these and included boulders or rock pieces less than two cubic yards in volume. These materials were removed with caterpiller tractors, and scrapers, front end loaders and road patrols. Wet materials were excavated with a dragline. Clay pockets were excavated with a backhoe and hand shovels. Shoring and supports were not required. Overburden materials were placed in the impervious zone, in berms, random or waste areas as directed. Overburden quantities totalled approximately 4,126,400 cubic yards as shown in Table 3 below.

Table 3

Overburden Excavation Quantities (1)

Area	Cubic Yards
Stage I	
Cutoff Trench	196,000
Foundation Excavation	376,000
Outlet Works	528,000
Stage II	
Foundation Sripping, Borrow and Approach Channel	1,100,000
Outlet Channel	1,170,000
Stage III	
Approach Channel	57,700
West Bulkheads	107,600
East Bulkheads	77,500
Outlet Channel	43,600
Stage VI	
Toe Trenches	25,000
Foundation Excavation	190,000
River Muck	60,000
Channel Plugs	195,000

Note(1) Sterett Creek Dike, Stage IV is not included.

5-04. Rock Excavation was defined as the removal of bedrock materials, (Cotter-Jefferson City dolomite limestone) between the lines and grades shown on the drawings regardless of how excavated, which were not included as "Excavation - Overburden". Tolerances for rock excavation were 3-inches ± for presplit surfaces and 6 inches for other surfaces.

CHAPTER 5

EXCAVATION PROCEDURES FOR COMPONENT PARTS

5-01. Excavation Grades: Overburden side slopes in the cutoff trench were cut 1V on 2H. Overburden side slopes in the approach and outlet channels and in the Spillway-Powerhouse excavation were cut 1V on 3H. The channel plugs were cut 1V on 3H. In the Spillway-Powerhouse area bedrock was excavated to El. 589 for the powerhouse foundation and to El. 606 for the spillway foundation. Along the upstream side of the powerhouse, a notch 45 feet wide and 6 feet deep was excavated down to El. 583. The downstream slope of the notch was cut 1V on 1H. A notch 40 feet wide and 10 feet deep was cut along the upsream side of the spillway. This notch extends to El. 596. The downstream slope was cut 1V on 0.7H. The approach channel was excavated from the structure on a slope of 1V on 3H up to El. 640. The outlet channel was excavated from the structure on a slope of 1V on 5H up to El. 630. All the faces of the Spillway-Powerhouse excavation were cut vertical. Side slopes of the approach and outlet channels were cut 1V on 0.75 and 1V on 2.5H. See Plates 39 and 44 thru 58.

5-02. Dewatering Provisions: Beginning with the Stage I Contract and continuing thru diversion and closure, the embankment and Spillway-Powerhouse areas were protected by construction of a perimeter dike. All excavation and fill placement except the downstream channel plug were performed in the dry. The foundation was dewatered by installing a system of well points inside the dike, parallel to the river, for a distance of 50 feet upstream and 50 feet downstream of the dam axis. Sand bag dikes, ditches, sumps and gasoline pumps were used to control seepage and rainwater. The excavations extended about 30 feet below river level (El. 750), in the cutoff trench and about 67 feet below river level in the powerhouse area. The first concrete placement on the project (19 Oct 71, see Photo 61), was for the sump on the left wall of the powerhouse service bay (Monolith 5), at El. 672, 78 feet below river level. During Stage II construction, the four 36-inch diameter calyx holes drilled in the Spillway-Powerhouse area were utilized as sumps to dewater the excavation. During closure in Stage VI, water between the upstream and downstream rock dikes was pumped out and the foundation was dewatered by a line of 14 deep wells drilled on the right bank of the river near Sta. 61+00, from 600 feet upstream to 500 feet downstream. See Plates 103 and 104, the holes were drilled by American Drilling Service Company using a Hughes, Model LDW-100 drill. A 22" diameter hole was drilled to rock, then reduced to 18" diameter for a depth of 10' into rock. A Johnson Water Well Screen 20' long, with 0.080 inch slot openings, attached to an 8" casing, was set in the bottom of each hole. Gravel pack was introduced against the screen and casing to within 35' of the top of the hole. The larger diameter "temporary" casing was removed and the balance of the well was backfilled to the top with concrete sand. Each well was cleaned by water intrusion for roughly 15 minutes at 1,000 gpm then pumped until discharge was reasonably clear. Then a 5hp Webral electric sumersible pump with a 200 gpm rating at 80' head, was installed in the bottom of each well and attached to a 4" riser. The 4" riser was attached to an 8" trunk line. Each well contained a 1 1/2" grout pipe later used to grout the system shut and for monitoring

angle of 75 degrees to the horizontal and the failure plane would exit at the junction of the base of the wall and the floor of the stilling basin. The design assumes the drainage system will relieve 50 percent or the hydrostatic pressure. Drain holes were drilled 2 feet deeper than the anchor bolt holes. See Plates 76 thru 78, 80 thru 84 and paragraph 10-3. The adopted spacing of rock anchor bolts was generally 6 feet on centers. On the spillway wall between El. 633 and 660, bolt lengths were 28 feet and spacing was 6 feet to 8.5 feet horizontal and 5 feet vertical. Below El. 633 spacing was 5 feet by 5 feet. On the powerhouse wall bolts were 23 feet long. Between El. 662 and 615.5 spacing was 6 feet, 8 inches horizontal and 5 feet vertical. Below El. 615.5, spacing was 5 feet, 6 inches horizontal and 5 feet vertical.

CHAPTER 4

SPECIAL DESIGN CONSIDERATIONS

4-01. Design Considerations: During the design stage, a meeting was held in the Kansas City District Office on 15 and 16 December 1964. Representatives of OCE, MRD and KCD were present. The following design features were discussed.

a. Slope Protection: Riprap designed according to EM 1011-2-2300 would cost about $400,000. It was agreed that if about 6 feet of rock from required excavation were used on the upper 1V on 3H slopes, without filter and 2 feet on the lower 1 on 10 slope, the cost would be greatly reduced. These concepts were incorporated in the design by requiring the large stone in Zone I rock be rock raked to the outer 20 feet of each lift.

b. Zoning: Preservation of rock excavation faces and utilization of required rock excavation were major considerations in the design of this project. The bulk of required rock excavation was utilized in the Zone I and Zone II rock fills. Embankment construction and rock excavation were scheduled to coincide in order that rock stockpiles would be reduced to the minimum required for rock dikes and other rock fills to be placed during and after closure.

c. Wrap Arounds: Impervious clay extends about 30 feet horizontally around the non-overflow bulkheads on the upstream and downstream sides. The clay is protected with an 8 feet wide zone of rock choker course and an 8 feet wide zone of scalped rock. On the downstream sides, the choker course ties in to the horizontal drain. See Plates 42 and 43.

d. Horizontal Drain: Design of the horizontal drain is a "first" for the Kansas City District. It utilizes a 6 feet thick layer of large rock (Zone II rock) sandwiched between thinner layers of finer rock, (2 feet layer of choker course on top and a 1.5 feet layer of scalped rock on the bottom). Large quantities of natural siliceous sand, normally used, are not available near the damsite. Crushed and graded Burlington limestone from Quarry Site 3 would also be expensive. The objection to use of limestone rock in drains is that they tend to break down during handling and compaction, producing fines that cement the rock together into a concrete like mass which prevents water from seeping through the drain. It was the opinion of the Kansas City District Office that this sandwich design would work and all parties agreed. See Plates 41 thru 43.

e. Design of Rock Bolting System was conducted by the Foundations and Materials Branch of the Kansas City District, US Army Corps of Engineers, under the direction of Mr. Jacob F. Redlinger. After a series of tests on selected samples of the rock, it was determined that the rock failure surface was at an angle of internal friction of 33 degrees. Cohesion intercept ranged from 0 to 50 tons per square foot. The design strengths were Q = 33 degrees and cohesion = 0. The analysis assumed a joint failure plane exists at an

along a potential failure plane. Based on these tests design strength of the weaker rock zones, parallel to bedding, was: tan Ø = 0.65 and cohesion = 10.8 k.s.f. Samples of breccia had shear strengths equal to or better than strengths of samples sheared along bedding planes. A conservative allowable bearing pressure for the foundation rock of 50 tons per square foot was selected. For complete test data see Design Memo No. 7, Spillway Design and Design Memo No. 3B, Volume I, Book I, Analysis of Design. All of the required rock excavation was used in construction of the embankment.

3-12. Unusual or Unanticipated Geologic Conditions Encountered During Construction: Early geologic investigations indicated possible existance at the damsite of extensive major faults, large sink hole structures, and extensive breccia zones. During excavation of the cutoff trench, a few small normal faults of small displacement were found. From Sta. 22+50 to Sta. 27+50, bedrock displayed prominent jointing. At Sta. 31+20 bedrock was intensely fractured. Several large clay filled solution channels or caves were found from Sta. 22+50 to Sta. 27+50. Large sink fill structures were encountered from Sta. 28+19 to Sta. 28+52 and from Sta. 39+37 to Sta. 40+68. All of the sinks solutioning and breccia zones were interformational and were more confined than had been anticipated. The faulting also was of more limited extent than had been anticipated. In the outlet channel from Sta. 57+50 to Sta. 64+94 (control line "B") and from Sta. 61+00 to Sta. 73+40 (control line "A"), the side slopes were flattened from IV on 0.75H to 1V on 2.5H because unexpected shale breccia in the bedrock would have caused the original slope to be unstable.

CHAPTER 8

FOUNDATION ANCHORS AND ROCK BOLTS

8-01 Foundation Anchors: Anchor bars were deformed No. 11 and No. 14S reinforcing steel conforming to ASTM Standard A 615 or A 617 grade 40. The anchor bars were grouted into 3 inch diameter vertical holes. The Specifications required the bars be installed within 48 hours after drilling the holes and not less than 48 hours prior to placing concrete. The grout consisted of one part portland cement, two parts job concrete sand, by dry weight, and sufficient water to obtain approved consistency. Spacers were fastened to each bar to center it in the hole and prevent contact with rock surfaces. The stilling basin slab and end sill is anchored with approximately 544, No. 11 anchor bars grouted 12 feet into bedrock and spaced 7 feet by 8 feet. Eight of the eleven baffles are each anchored with 8 No. 14S bars grouted 35 feet into bedrock and 4 No. 11 bars grouted 16 feet into bedrock. The three remaining baffles are anchored to the reinforcing steel of the divider wall base slab. See Plates 76, 77 and 78.

8-02. Rock Bolts were deformed No. 11, 1 3/8 inch diameter, reinforcing steel conforming to ASTM Standard A 615 grade 75. One end was threaded for 12 inches at 12 threads per inch. The rock bolts were used: (1) To control stress relief, (2) to strengthen the rock faces, (3) to anchor the concrete walls, (4) to support draped fencing and (5) to repair damaged rock corners or unstable rock faces. All rock bolts were installed within 24 hours after blasting final faces previously formed by prespliting. Two types were used. See Plates 79 thru 84.

a. Short Perforated Sleeve Bolt: These bolts were used to strengthen the vertical rock faces in the Spillway-Powerhouse structure area. Installation was as follows: A 2 1/4 inch diameter hole was drilled horizontally into the rock face to the required depth and washed with water. A short (5 feet long), 2 inch diameter, steel perforated sleeve was filled with quick-set mortar and pushed to the back of the hole. The bolt was then inserted and driven into the mortar-filled sleeve with an air powered hammer, forcing mortar through the perforations and into intimate contact with the bolt, sleeve and sides of the hole. Small plastic, grout and vent pipes, previously taped to the bolt, were threaded through a quick-set mortar plug and leveling pad. A 12 inch by 12 inch by 1/2 inch steel bearing plate, with holes for the grout and vent pipes, was placed over the threaded bolt and secured with a steel bevel washer, a hardened steel washer, and heavy duty hex nut. After the mortar had set for a minimum of 8 hours, the bolt was tensioned with a hydraulic jack to 47,000 pounds. The hex nut was then torqued until the jack load was transferred to the hex nut. The bolts were retentioned and grouted after blasting within 100 feet, in any direction, had been completed and at least 48 hours prior to placing concrete which covers the bolt, coupler and extension. Grout was injected at 10 p.s.i. When grout emitted from the vent pipe, the vent pipe was plugged and grouting stopped. A typical grout batch consisted of 188 lbs. portland cement, 75 lbs. fly ash, 2.6 lbs. Intraplast-C and minimum amount of water to produce a pumpable mix. Quick-set

mortar consisted of 2 parts portland cement, one part job concrete sand (100 percent finer than No. 10 sieve), accelerator (Sika Set), and minimum amount of water to make a workable mix to set within 8 hours. In the early part of the work, bolt holes were drilled 2 1/4 inch diameter but later, at the Contractor's request, the holes were drilled 3 inch diameter. Also, at the Contractor's request, two short sleeves were used in some of the holes where soft shale or breccia made anchorage difficult.

b. Full Perforated Sleeve Bolts: These bolts were used to strengthen the excavated rock faces and to firmly fasten the concrete walls to bedrock and create a concrete and rock gravity mass able to withstand the anticipated hydrostatic pressure on these walls. These bolts were similar to the short sleeve anchored bolt except the sleeve extends for the entire length of the bolt and the steel bearing plate was 6 inch by 6 inch by 1/4 inch thick. Anchorage is throughout the hole and tensioning and grouting were not required. Table 6 below shows all rock bolts installed during the Stage III Contract except 41 rock bolts installed between Monoliths 4 and 5 which were installed by Contract Modification.

Table 6

Rock Bolts Installed

Bolt Length in feet	Number of Bolts	Total Length in feet
11	399	4,389
16	76	1,216
23	452	10,396
28	283	7,924
	1,210	23,925

Extra bolts installed under the direction of the Project Geologist:

Bolt Length in feet	Number of Bolts	Total Length in feet
8	2	16
11	10	110
16	4	64
23	2	46
26	46	1,196
28	21	588
	85	2,020

Table 7 below shows the number of rock bolts which pulled out or were termed failures by the Project Geologist and were replaced with an additional bolt.

Table 7
Replacement Bolts

Bolt Length in feet	Number of Bolts	Total Length in feet
11	6	66
16	5	80
23	17	391
28	19	532
	47	1,069

c. Bolt Tensioning: The equipment used to tension the rock bolts consisted of a Simplex RP 8A, Two Way Hand Pump Hydraulic Jack with maximum force of 10,000 p.s.i., an RC 613 OTC (Owatanna Tool Corp.) Power Twin Ram Jack of 30 tons capacity and a gage to reflect load. The short sleeve bolts were tensioned to 47,000 pounds with the hydraulic jack. The hex nut was then torqued until a drop of 2,000 pounds was noted on the gage and the jack was removed. During retensioning the amount of original tension remaining on the bolt was noted. Table 8 below shows the percent of initial tension remaining on the bolts at the time of retensioning.

Table 8

Percentage of Initial Bolt Tension Prior to Retensioning

Powerhouse or West Side Wall

Initial Tension Kips	Powerhouse Tailrace TW	Powerhouse Wall	Powerhouse Approach Wall	Average
39.2 - 47.0	52	63.4	62.5	59.3
32.6 - 37.9	21.4	18.3	12.5	17.4
26.1 - 32.3	16.9	11.8	9.4	12.7
19.6 - 24.8	6.9	3.2	6.3	5.5
Less than 19.6	2.8	3.2	9.4	5.1

Spillway or East Side Wall

Kips	Spillway TW	Spillway Structure	End of Spillway TW	Average
39.2 - 47.0	71	60.5	72.5	68.0
32.6 - 37.9	11.0	11.1	11.5	13.2
26.1 - 31.3	10.3	13.6	5.0	9.6
19.6 - 24.8	6.6	4.9	5.0	5.5
Less than 19.6	1.0	9.9	0	3.6

8-03. Deformeter Bolts were long shell, No. 8 HR, Deformeter Type R-1 manufactured by Williams Engineering Corp., Grand Rapids, Michigan. A deformeter rock bolt is a special type of hollow rock bolt. The hollow space contains a small diameter steel rod, which is welded to the distal (anchorage) end of the bolt and extends to the outside. When the bolt is tensioned and elongated the inner rod appears to recede. When bolt tension is relaxed the rod appears to extend from the bolt. Anchorage is achieved by an expanding wedge at the distal end of the bolt. The bolts were tensioned to 20,000 pounds using a torque wrench and retensioned twice more before they were grouted.

Twenty-three deformeter bolts were installed along the final faces of the rock excavation in order to measure free relaxation and restrained relaxation forces of the rock strata. Table 9 shows location of the bolts.

Except for Bolt 1, which was not properly documented, each bolt was installed, seated, and read with an Ames gage immediately before tensioning, after tensioning, and periodically thereafter. Inherent tension was calculated as follows in order to ascertain a pattern or figure of strain transcribed to the bolt by the rock.

The Modulus of Elasticity is a constant expressing the ratio between unit stress and unit deformation.

$$E = \frac{S}{e} \quad \text{where } S = \frac{P}{A} \quad \text{and } e = \frac{d}{1}$$

when E = Modulus of elasticity
P = axial force
A = cross sectional area of the rod
S = unit stress produced by force P
d = deformation produced by the force P of length 1.

In this formula the axial force is determined and deformation is the measured quantity. Modulus of Elasticity is 29 Million p.s.i. Table 10 shows the strain and axial force exhibited by the deformeter bolts. An attempt was made to ascertain a pattern or directional relationship of the inherent rock stress or a relationship between axial force and bolt elevations. No relationships were recognizable. Theoretically the deformeter bolts should have provided information of positive value. Force trends or strain in the rock as shown in Table 10 are considered to be the result of slippage of the deformeter's anchorage plus adverse effects of blasting.

Table 9

Deformeter Bolt Locations

Bolt Number	Station	Length	Range	Elev.	Structure
1.	49+30	15'	3+00R	650	Powerhouse Approach Wall
2.	52+60	15'	2+30L	640	Spillway Training Wall
3.	51+30	15'	3+05R	642	Powerhouse Wall
4.	49+95	15'	2+55L	634	Spillway Structure
5.	49+95	15'	2+55L	617	Spillway Structure
6.	51+05	15'	2+25L	615	Spillway Training Wall
7.	51+95	15'	2+25L	615	Spillway Training Wall
8.	49+95	15'	2+95R	620	Powerhouse Wall
9.	51+30	15'	3+05R	622	Powerhouse Wall
10.	50+50	20'	2+90R	605	Powerhouse Wall
11.	52+75	20'	2+90R	620	Tailrace Training Wall
12.	51+85	20'	2+85R	605	Tailrace Training Wall
13.	51+30	15'	3+05R	602	Powerhouse Wall
14.	49+10	20'	2+90R	610	Powerhouse Approach Wall
15.	49+80	20'	2+90R	595	Powerhouse Wall
16.	51+16	20'	3+00R	592	Upstream Wall Erection Bay Sump
17.	49+95	20'	0+43L	595	Spillway Structure
18.	49+45	15'	2+35L	615	Approach Channel Rock Wall
19.	49+45	15'	0+75L	605	Approach Channel Rock Wall
20.	49+60	15'	0+90R	590	Approach Channel Rock Wall
21.	51+50	20'	3+00R	593	Section HH Right Non-Overflow
22.	52+80	20'	0+25L	600	End Sill
23.	51+60	15'	Divider Wall		Vertical Installation
24.	49+60	15'	150R		Vertical Installation

Table 10

STRAIN AND DEVELOPED AXIAL FORCE EXHIBITED BY DEFORMETERS

Bolt	d	a	d-a	e	+	(-)	Axial Force Lbs.
2	0.169	0.122	0.047	0.004	0.051	0.016	1900
3	0.169	0.120	0.049	0.043	0.092	0.028	3320
4	0.169	0.101	0.068	0.010	0.078	0.007	830
5	0.225	0.122	0.103	0.003	0.106	0.020	1780
6	0.169	0.140	0.029	0.015	0.044	0.024	2850
7	0.169	0.101	0.068	0.007	0.075	0.024	2850
8	0.169	0.051	0.118	0.000	0.118	0.016	1900
9	0.169	0.108	0.061	0.016	0.077	0.004	470
10	0.225	0.034	0.191	0.000	0.191	0.012	1070
11	0.225	0.156	0.069	0.027	0.082	0.009	800
12	0.225	0.062	0.163	0.003	0.166	0.028	2490
13	0.169	0.079	0.090	0.035	0.125	0.020	2370
14	0.225	0.164	0.061	0.011	0.072	0.030	2670
15	0.225	0.077	0.148	0.021	0.169	0.020	1780
16	0.225	0.070	0.155	0.000	0.155	0.023	2050
17	0.225	0.063	0.162	0.005	0.167	0.028	2490
18	0.169	0.116	0.053	0.005	0.058	0.011	1300
19	0.169	0.084	0.085	0.006	0.091	0.019	2260
20	0.169	0.111	0.058	0.008	0.066	0.022	2610
21	0.225	0.010	0.215	0.003	0.218	0.006	530
22	0.169	0.045	0.124	0.002	0.126	0.017	2020
23	0.169	0.025	0.144	0.007	0.151	0.036	4270
24	0.169	0.030	0.139	0.010	0.149	0.011	1300

Legend:

d = Deformation d in inches at Unit stress S when tensioned to 20,000 Lbs
a = Measured elongation in inches after tensioning
d-a = Loss in inches by plate deformation or consolidation by the rock
e = Added consolidation in inches to point of retension
+ = (d-a) + e
(-) = elongation in inches after maximum consolidation

CHAPTER 9

CHARACTER OF FOUNDATION

9-01. Foundation Surface:

a. Left Abutment: At the toe of the left abutment about Sta. 70+20 bedrock exposed in the cutoff trench was Cotter-Jefferson City Formation Unit 12. It is medium bedded to massive, moderately hard, finely crystalline mottled blue-gray and tan. Oolitic chert bands occur near the middle and intraformational conglomerate was encountered. Proceeding up the abutment, shale breccia extends from Sta. 70+20 at El. 622 to Sta. 72+00 at El. 729. The upper one to two feet of the breccia was stripped off to expose firm bedrock. At Sta. 72+00, El. 729 to El. 730, Sylamore Shale was encountered. it was conglomeratic, massive and soft to moderately hard. From Sta. 72+00 at El. 730 to Sta. 74+81, end of cutoff trench, at El. 767, Sedalia-Compton limestone was encountered. It is thin to medium bedded, with thin shale bands and partings and chert nodules. From El. 751, Sedalia-Compton limestone was excavated on a stair step slope 1V on 1H down to about El. 730, and a minum berm width of 15 feet was provided to protect the underlying shale breccia. See Plate 93.

b. Spillway-Powerhouse: The foundation surface of the Spillway-Powerhouse is fairly flat-lying but somewhat irregular. Bedrock units forming the foundation surface range from Unit 15 at El. 589.5, to Unit 18 at El. 583.0. For descriptions see Plates 59, 60 and 61. For map of powerhouse foundation see Plate 64.

9-02. Condition of Foundation Soil or Rock: This item is covered in above paragraphs 3-11, 3-12, 5-09 (a), (b), and (c).

9-03. Water: This item is covered in above paragraphs 3-09, 5-02, and 6-01.

9.04. Special or Unusual Conditions: No special or unusual conditions were encountered in construction of the project except the design of the downstream horizontal drain described in paragraph 4-01(d).

CHAPTER 10

FOUNDATION TREATMENT

10-1. Grouting Prior to Concrete Placement:

a. Solution Cavities in Spillway-Powerhouse Foundation: During cleanup of the Powerhouse foundation, 7 October, 1971, solution cavities were found between El. 583 and El. 589 in Unit 17-B of the Cotter-Jefferson City formation. The presence of these cavities indicated that there could be a problem in affecting a water seal if the cavities were interconnected solution channels running through the grout curtain. The cavities could hardly be called solution channels as they appeared lenticular and locally developed. They occurred in sandy textured dolomite in conjunciton with joint traces. Orientation was not transversely across the curtain, but more on east-west planes parallel to the dam axis.

Each solution cavity was "broken out", excavated, and explored to determine its limits. Shallow cavities with no apparent extensions were backfilled with concrete. Deeper cavities (generally over two feet) or cavities with indeterminable limits were explored where possible to the extent which permitted a grout and vent tube installation. The installation was subsequently grouted. Pressure grouting of all possible avenues of potential communication, sealed the area beyond a reasonable doubt. See Plate 64 for location of grout holes in the Powerhouse foundation. Table 11 below shows the location and grout takes. A total of 36.98 sacks of grout was injected into 19 holes.

Table 11

Grouting of Solution Cavities in Spillway-Powerhouse Foundation

Grout hole location	(Sta. & Range from Control Line C)			Sacks
Station	Range	Elevation	Date Grouted	Injected
49 + 82	1 + 75 R		10/26/71	8.35
50 + 05	1 + 75 R on 1 to 1 slope		10/26/71	3.22
49 + 64	2 + 88 R		10/26/71	0.14
50 + 05	1 + 40 R on 1 to 1 slope		10/27/71	0.57
50 + 05	1 + 00 R on 1 to 1 slope		10/27/71	0.73
50 + 05	0 + 90 R on 1 to 1 slope		10/27/71	0.00
50 + 05	0 + 80 R on 1 to 1 slope		10/27/71	3.14
50 + 60	0 + 45 R on 589 Bench		10/28/71	8.21

Table 11 --Continued

Grouting of Solution Cavities in Spillway-Powerhouse Foundation

Grout hole location Station	Range	(Sta. & Range from Control Line C) Elevation	Date Grouted	Sacks Injected
50 + 60	0 + 43 R on 589 Bench		10/28/71	0.20
50 + 60	1 + 05 R on 589 Bench		10/28/71	3.20
50 + 65	1 + 10 R on 589 Bench		10/28/71	0.10
50 + 65	1 + 13 R on 589 Bench		10/28/71	0.23
49 + 85	2 + 88 R	591.5	3/30/72	0.50
49 + 80	2 + 88 R	591.5	3/30/72	0.25
51 + 10	1 + 98 R	589	5/08/72	0.04
51 + 65	0 + 43 L	592	5/24/72	8.1
50 + 49	0 + 33.75 L	589		
50 + 42	0 + 42.75 L	589		
51 + 10	0 + 80 R	589	6/09/72	0.00
			TOTAL	36.98

b. Additional Curtain Grouting Spillway-Powerhouse Foundation: After discovery and grouting of the solution cavities in the Spillway-Powerhouse foundation it was decided that additional curtain grouting further upstream should be done before placing concrete. A series of 66 primary holes, 22 feet deep, on 5-foot centers and angled 20 degrees to the right, were drilled and grouted along a line 32 feet upstream of the dam axis. The additional grout holes extend from Sta. 40+71.25 to Sta. 43+96.25.* The grout holes are shown on Plates 130, 131 and 132 and a summary of grout takes is shown in Table 12 below. A total of 27.07 sacks of grout was injected.

*Note: Records are incomplete. It is believed that 36 additional grout holes were also drilled and grouted on a line 9 feet upstream from the dam axis from Sta. 44+11.25 to Sta. 46+01.25 but no grouting records of these holes has been found.

Table 12

Summary of Additional Curtain Grouting Spillway-Powerhouse

Grout Hole Locations		Sacks	Grout Hole Locations		Sacks
Dam Station	Depth in feet	Injected	Dam Station	Depth in feet	Injected
40 + 71.25	21.0	1.00	42 + 41.25	21.5	0.24
40 + 76.25	22.0	2.50	42 + 46.25	21.5	0.50
40 + 81.25	20.0	0.15	42 + 51.25	21.5	0.99
40 + 86.25	20.0	3.05	42 + 56.25	21.3	0.24
40 + 91.25	20.0	0.11	42 + 61.25	21.3	0.10
40 + 96.25	22.0	0.00	42 + 66.25	21.3	0.10
41 + 01.25	22.0	0.06	42 + 71.25	21.3	0.40
41 + 06.25	23.0	0.00	42 + 76.25	21.4	0.90
41 + 11.25	22.0	1.35	42 + 81.25	22.5	0.66
41 + 16.25	23.0	2.10	42 + 86.25	22.5	0.40
41 + 21.25	22.0	0.06	42 + 91.25	22.5	0.21
41 + 26.25	23.0	0.12	42 + 96.25	21.0	0.20
41 + 31.25	23.0	0.00	43 + 01.25	22.0	0.19
41 + 36.25	23.0	0.29	43 + 06.25	22.5	0.21
41 + 41.25	21.0	0.11	43 + 11.25	22.3	0.14
41 + 46.25	22.5	0.30	43 + 16.25	22.0	0.00
41 + 51.25	21.0	0.17	43 + 21.25	22.5	0.12
41 + 56.25	22.5	0.57	43 + 26.25	22.5	0.11
41 + 61.25	21.2	0.17	43 + 31.25	22.5	0.14
41 + 66.25	22.5	0.36	43 + 36.25	22.5	0.00
41 + 71.25	21.5	0.17	43 + 41.25	21.5	0.13
41 + 76.25	22.5	0.00	43 + 46.25	22.5	0.03
41 + 81.25	21.0	0.30	43 + 51.25	22.5	0.00
41 + 86.25	22.5	1.20	43 + 56.25	22.5	0.10
41 + 91.25	21.0	0.60	43 + 61.25	21.5	0.21
41 + 96.25	22.5	0.60	43 + 66.25	21.5	0.68
42 + 01.25	21.0	0.23	43 + 71.25	21.5	0.57
42 + 06.25	22.5	0.13	43 + 76.25	22.5	0.03
42 + 11.25	21.5	1.12	43 + 81.25	22.5	0.03
42 + 16.25	22.5	0.49	43 + 86.25	22.5	0.09
42 + 21.25	22.5	0.80	43 + 91.25	22.5	0.20
42 + 26.25	23.0	0.00	43 + 96.25	22.5	0.59
41 + 31.25	21.7	0.14	TOTALS	148.5	27.07
41 + 36.25	23.0	0.06	Sack/Lineal Foot = 0.182		

10-2. Curtain Grouting. In order to minimize water seepage through the
rock a continuous grout curtain was constructed across the valley from the
nt to the left abutment (Sta. 22+38 to Sta. 74+81). In addition to the
ıt curtain 3-inch diameter drain holes were drilled downstream of the
tain in the spillway-powerhouse section. The grout holes were drilled
allel to the dam axis and inclined landward 30 degrees from vertical except
the spillway-powerhouse area where holes were angled in various directions.
mary holes were drilled on 12-foot centers. After drilling was completed
n grout hole was washed with clear water, pressure tested and grouted
ough a rubber packer. The amount of water pressure and grout pressure
lied was only that amount required to balance the rock pressure at the
ker depth. Grouting of each hole usually began with a thin mix, (one part
ent to 4 or 5 parts water), and if a significant amount of thin grout was
ected, the grout mixture was thickened. Packer depths, water and grout
ssures and takes are shown on the Grout Curtain Profiles. All of the
uting was directed and supervised by a government inspector. Complete
ords of each grout hole were maintained. A complete summary of grout takes
each stage of the grout curtain is shown on Plate 153. The amount of
ut injected into bedrock was less than had been anticipated. The overall
rage for all grout holes was 0.0198 sacks per lineal foot. Plate 153A
ws pounds of cement injected per lineal foot of grout hole for each section
the grout curtain. Forty-three percent of all grout was injected into 259
es in the deepest section of the grout curtain under the spillway-
erhouse during stage III construction.

a. Stage I Construction. Drilling and grouting began 27 July 1966 and
completed 20 December 1966. Grouting was performed from the floor of the
off trench on bedrock. Three-inch diameter holes were drilled 1 foot into
lrock and steel casings were cemented in. Grout holes were drilled with
.l mounted Chicago Pneumatic drills and 2 1/2-inch diameter non-coring bits.
ie holes were drilled with precussion drills. Grout was mixed in a double-
mber mixer with agitator sump with a capacity of 20 cubic feet. Grout was
ıped with a Moyno pump. The drilling and grouting equipment was air-driven
.ng a Chicago Pneumatic #600 air compressor. The stop grouting method was
d. Grout holes were drilled to total depth, then washed, pressure tested
l pressure grouted through a packer which was set at successive shallower
ths and lower pressures. Zone I extends from 0 to 16 feet. Zone II
ends from 16 to 40 feet and Zone III extends from 40 to 80 feet. Primary
.es were drilled on 12-foot centers. The grout curtain extends from Sta.
38 to Sta. 38+00 and from Sta. 48+36 to Sta. 63+36. Grout takes were small
ept in the area of a sink structure from Sta. 28+19 to Sta. 28+52. See
ıte 110. Pressures, depths and takes are shown on Plates 106 thru 125. Two
ıdred and ninty six primary holes took 88.06 sacks in 14,480 lineal feet.
hundred and sixty secondary holes took 43.90 sacks in 8,278 lineal feet
536 tertiary holes took 14.85 sacks in 6,067 lineal feet.

b. Stage III Construction. During Stage III Construction the grout
tain was constructed through the spillway-powerhouse area. Drilling and
uting began 10 November 1973 and was completed 10 June 1974 and was
formed from inside the grouting gallery prior to drilling of drain holes
after concreting of the spillway-powerhouse structure and non-overflow

bulkhead sections had been completed. See Plates 126 thru 135. Zone I extends from 0 to 40 feet. Zone II extends from 40 feet to 65 feet and Zone III extends from 65 feet to 110 feet. The grout curtain extends from Sta. 37+75 to Sta. 48+84 and overlaps the Stage I Construction grout curtain at each end. Primary holes were drilled on 12-foot centers. Most of the grout holes were vertical and parallel to the dam axis but angled holes were also drilled and grouted. Grout takes were higher than in Stage I Construction especially in monoliths 6, 7 and 8 which took 4 times more grout. Water flowed from drilled holes before grouting. The largest flow was from between secondary holes S-8 andS-9. The water was believed to be coming from hole depths 40 to 70 feet (El. 520 to El. 550). One hundred and thirty-four primary holes took 357.19 sacks in 9,581 lineal feet. Eighty-nine secondary holes took 186.70 sacks in 5,101 lineal feet and 35 tertiary holes took 47.94 sacks in 1,674 lineal feet. One quaternary hole took no sacks in 71 lineal feet.

c. Stage IV Construction. The right abutment of Sterett Creek Dike was grouted from the floor of a cutoff trench from Sta. 9+70 to Sta. 11+38. Primary holes were drilled on 12-foot centers. The holes were inclined toward the abutment 30 degrees from vertical and parallel to the dike axis. Zone I extends from 0 to 10 feet. Zone II extends from 10 feet to 28 feet and Zone III extends from 28 feet to 60 feet. Twenty sacks of grout were injected into 30 holes. See Plate 135. Grouting was performed in June 1975.

d. Stage VI Construction. During Stage VI Construction the Grout curtain was constructed from Sta. 63+36 to Sta. 74+81. Drilling and grouting began 6 October 1977 and was completed 28 July 1978. Three grout lines were constructed. Line A is 10 feet downstream and line C is on the dam axis. Line B is 10 feet upstream of the dam axis. The holes were drilled parallel to the dam axis and inclined landward 30 degrees from vertical. Primary holes were spaced on 12-foot centers. Drilling and grouting of line C was maintained 50 feet behind drilling and grouting of lines A and B and line C extends to a greater depth. Pressures, depths and grout takes are shown on Plates 136 thru 152. Grout holes were drilled with rail mounted Chicago Penumatic Model 55 drills using 1 5/8-inch non-coring diamond bits. Grout was mixed in a 20 cubic feet agitator mixer. For lines A and B Zone I extends from 0 to 15 feet and Zone II extends from 15 feet to 30 feet. For line C Zone I extends from 0 to 5 feet. Zone II from 5 to 15 feet. Zone III from 15 to 35 feet and Zone IV from 35 to 60 feet. The largest takes and grout leaks at the surface were in shale and sandstone breccia from Sta. 70+20 to Sta. 71+10 on line C. A large buried block of dolomite was encountered from Sta. 71+20 to Sta. 71+50. Dolomite breccia covers the block. The dolomite breccia was tighter and stronger than had been expected. The A and B lines were drilled and grouted first and were tighter at the surface than line C which was drilled and grouted last. The surface was soaked with water when drilling and grouting began on line C. Two hundred and sixty-two primary holes took 209.72 sacks in 11,379 lineal feet. Two hundred and fifty-seven secondary holes took 253.31 sacks in 8,078 lineal feet. Two hundred and thirty-three tertiary holes took 166.27 sacks in 2,659 lineal feet and 16

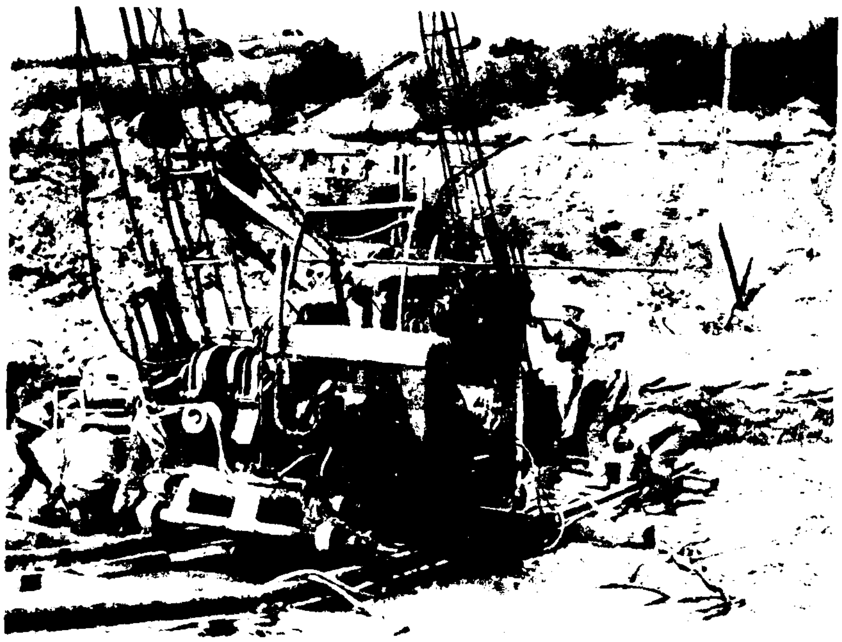
ıary hoıes took 1.31 sacks in 181 lineal feet. Most of the grout leaks ed in the top weathered 2 feet of bedrock which was later excavated final cleanup. From Sta. 72+00 to Sta. 74+81 bedrock is the Sedalia-n Formation. No grout was injected. Total drilling was 22,297 lineal Grout injected totaled 618.63 sacks and backfill totaled 901 sacks. A of 2,389 pounds of bentonite was added to the grout.

3. Drainage Provisions. Three-inch diameter drain holes were drilled floor of the spillway-powerhouse grouting gallery. The holes were el to the dam axis on 10-foot centers and angled downstream 20 degrees ertical to El. 540. Addition drain holes were drilled horizontal and at s angles at the ends of monoliths 6, 9 and 13. See Plates 126 thru 135. mps with a capacity of 250 gpm each were installed to pump out the ge water. Space was provided for one more pump if needed. Piping with city of 1,200 gpm was installed. For design purposes the drains were d to be 50 percent effective and the estimated seepage at pool El. 706 g.p.m. at pool El. 739.6, 575 g.p.m. On the powerhouse training wall, diameter drain holes were drilled 3 degrees above horizontal and 2 feet than the rock anchor bolts. The drain holes were spaced approximately H by 5 feet V. On the spillway training wall, drain holes were spaced imately 6 feet H by 5 feet V. In the stilling basin floor, drain holes nclined upstream 30 degrees from vertical and drilled to a depth 2 feet than the anchor bars. The drains were spaced approximately 7 feet by 7 See Plates 77 and 78.

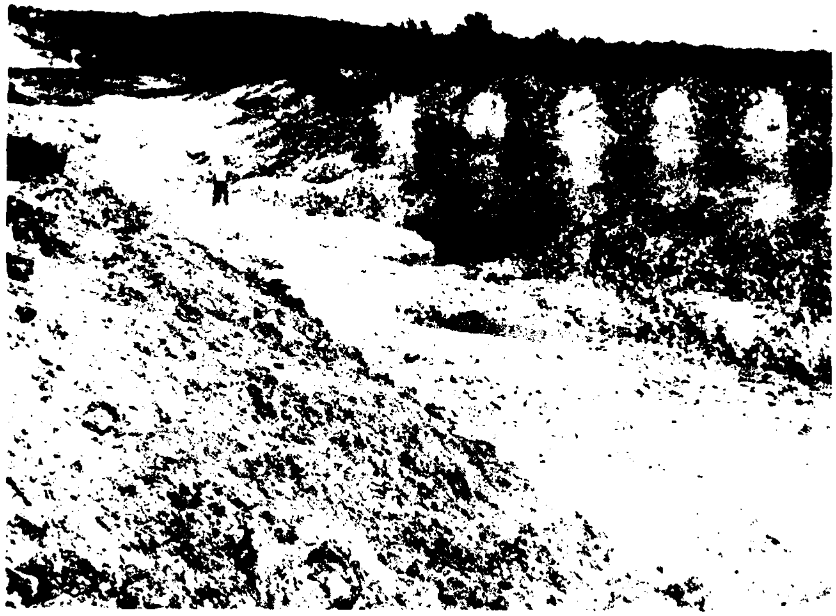
4. Foundation Compaction or Consolidation. This item is covered in aphs 5-09 (a) and (b).

5. Dental Concrete and Gunite. Several areas in the Spillway-ouse excavation were overexcavated and dental concrete was required. on 1H downstream slope of the powerhouse foundation notch required ill concrete. See Plate 64. The left wall and floor of erection bay th 5 required backfill concrete. See Plate 63. Gunite and wire mesh equired on the upstream side of the left wall of the spillway ation. See Plate 70. Backfill concrete was required at outside corners avated rock walls and at the junction of top of rock and the spillway werhouse training walls. See Plates 73 and 74. Concrete and gunite equired to repair overbreaks and damaged corners on the 1V on .75H at El. 634 bench and El. 632 bench on the upstream side of Monoliths 9 3. See Photos 165 and 166. In the cutoff trench from Sta. 22+50 to Sta. , Karst features on the bedrock surface were cleaned out and grouted See Plate 116. Caves on the sides of the cutoff trench were cleaned d backfilled with concrete. See Photos 26 thru 32.

21. Harry S. Truman Dam, 29 August 1966, Neg. No. 82412. Cutoff Trench Sta. 26+24 looking downstation. Pinnicales after blasting cavity at Sta. 25+00.



22. Harry S. Truman Dam, 30 August 1966, Neg. No. 82413. Cutoff Trench Sta. 62+80 looking upstation drilling reverse hole at Sta. 63+36.



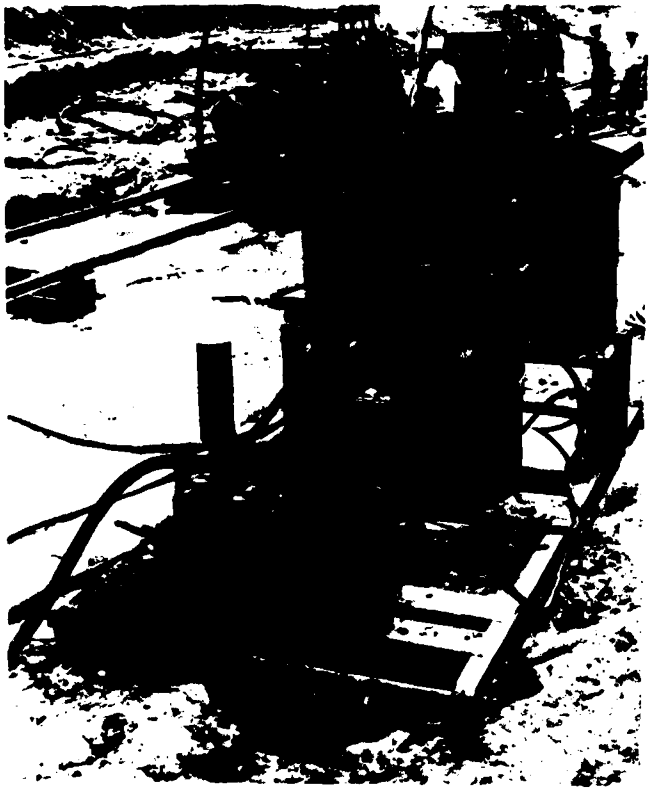
19. Harry S. Truman Dam, 26 August 1966, Neg. No. 82431.
Cutoff Trench Sta. 24+25. Looking upstation.

20. Harry S. Truman Dam, 26 August 1966, Neg. No. 82432.
Cutoff Trench Sta. 28+23. Looking downstation.

17. Harry S. Truman Dam, 25 August 1966, Neg. No. 82426. Cutoff Trench Sta. 58+10. Looking upstation.

18. Harry S. Truman Dam, 25 August 1966, Neg. No. 82430. Cutoff Trench Sta. 60+50. Looking downstation.

15. Harry S. Truman Dam, 27 July 1966, Neg. No. 965-64. Cutoff Trench Sta. 58+50. Looking upstation. Drilling grout holes at Sta. 58+92 and Sta. 59+16.

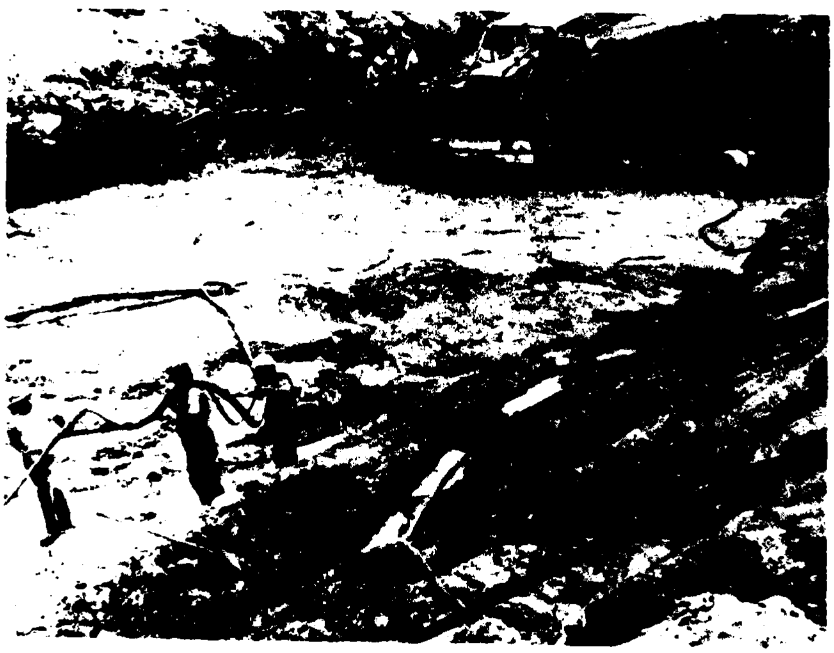


16. Harry S. Truman Dam, 27 July 1966, Neg. No. 965-94, Cutoff Trench Sta. 58+92 grout pump and mixer.

13. Harry S. Truman Dam, 27 July 1966, Neg. No. 965-63.
Cutoff Trench Sta. 55+70 looking upstation.

14. Harry S. Truman Dam, 27 July 1966, Neg. No. 965-8.
Cutoff Trench Sta. 56+50 looking downstation.

11. Harry S. Truman Dam, 27 July 1966, Neg. No. 965-117.
Cutoff Trench Sta. 52+80 looking upstation.



12. Harry S. Truman Dam, 27 July 1966, Neg. No. 965-120.
Cutoff Trench Sta. 54+80 looking downstation.

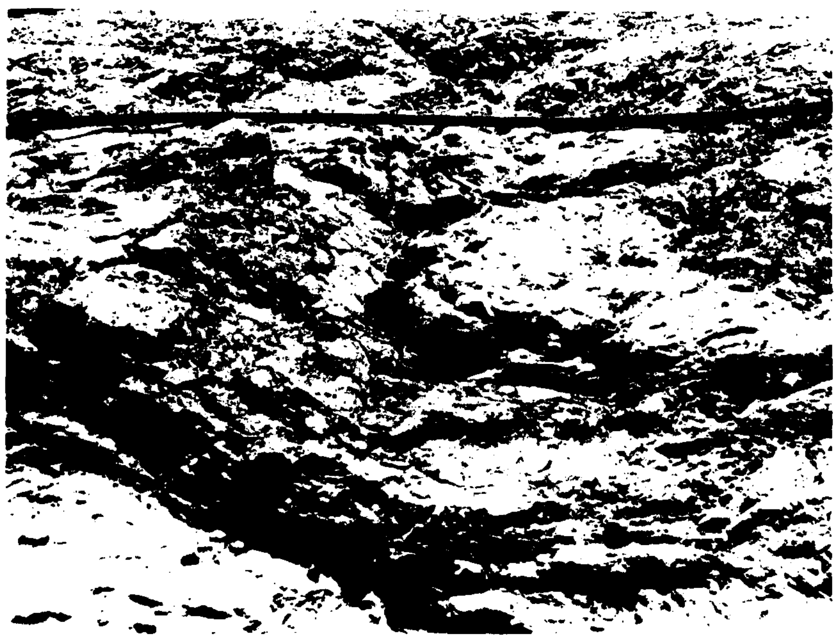
9. Harry S. Truman
27 July 1966,
Neg. No. R-245-1
Cutoff Trench
Sta. 48+50. Loo
upstation placin
previous and imp
vious fill.

10. Harry S. Truman Dam, 27 July 1966, Neg. No. R-245-20. Cutoff Trench Sta. 51+60 to Sta. 51+90, looking Upstation.

7. Harry S. Truman Dam, 27 July 1966, Neg. No. R-245-9. Cutoff Trench Sta. 51+15. Looking downstation placing pervious and impervious fill.

8. Harry S. Truman Dam, 27 July 1966, Neg. No. R-245-12. Cutoff Trench Sta. 51+90. Looking upstream. Foundation cleanup.

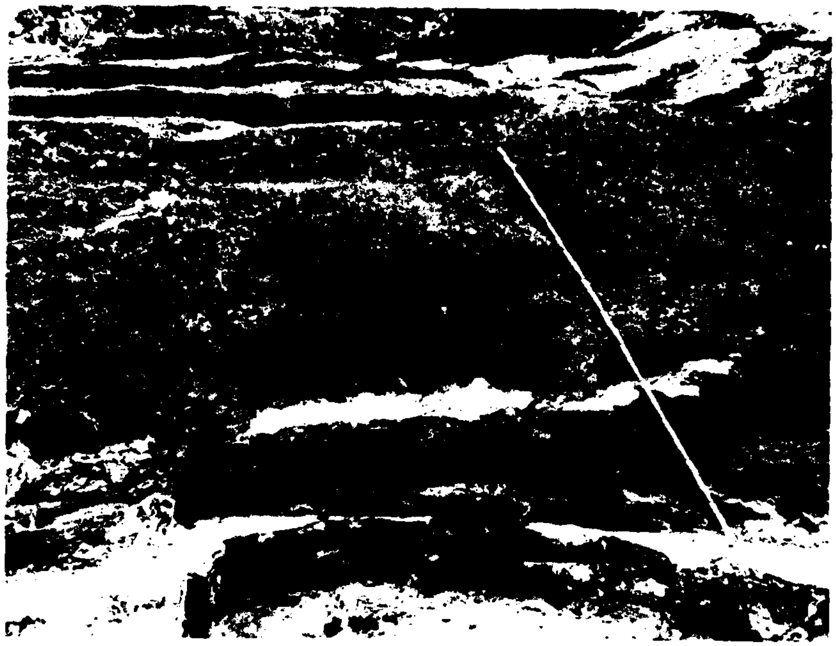
5. Harry S. Truman Dam, 26 July 1966, Neg. No. R-244-16. Cutoff Trench Sta. 50+25. Looking downstream.



6. Harry S. Truman Dam, 27 July 1966, Neg. No. R-245-4. Cutoff Trench Sta. 50+80 to Sta. 51+00. Looking downstream.

3. Harry S. Truman Dam, 26 July 1966, Neg. No. R-244-12. Cutoff Trench Sta. 49+70 to Sta. 50+45. Looking downstation from downstream side.

4. Harry S. Truman Dam, 26 July 1966, Neg. No. R-244-14. Cutoff Trench Sta. 50+00. Looking downstation from downstream side.



1. Harry S. Truman Dam, 26 July 1966, Neg. No. R-244-4. Cutoff Trench Sta. 49+20, Looking downstation on downstream side. Brown dolomite breccia overlain by shale.

2. Harry S. Truman Dam, 26 July 1966, Neg. No. R-244-7. Cutoff Trench Sta. 50+23. Looking downstation from upstream side.

PHOTOGRAPHS

PHOTOGRAPHS

CHAPTER 12

POSSIBLE FUTURE PROBLEMS

12-01 Conditions That Could Produce Problems: In May 1981 Periodic Inspection No. 2 was conducted. The following conditions which could cause future problems were noted:

a. Left Bank of Outlet Channel. As noted on previous inspections the left bank of the outlet channel immediately downstream of the spillway training wall is continuing to erode. The rock units are thin bedded, jointed and fractured. During periods of high flows loose rock fragments are plucked out and carried downstream. A study is underway to determine how to repair this rock face and prevent further erosion.

b. Downstream Embankment. A wet area was observed over the entire surface of the closure section below the interceptor ditch at the toe of the slope. The slope was saturated at the surface with occasional small areas of standing water. Saturation extended 3 to 5 inches below the surface. This condition may be the result of recent rains but the area will be continuously checked especially during dry periods, to determine if the condition persists. Several seeps and pin boils were observed at the downstream toe near the left abutment. The largest boil was located about 30 feet from the grouted gutter at the downstream toe and clear water was flowing freely.

c. Left Abutment Upstream Clay Blanket. Wave action has eroded the left abutment at a point about 3,000 feet upstream of the dam. This erosion has exposed underlying shale breccia and may increase the amount of seepage water entering the breccia. If left abutment seepage increases, the exposed shale breccia may require grouting to prevent entry of lake water.

12-02. Recommended Observations: Erosion of the left bank of the outlet channel and seepage conditions of the left abutment and downsteam closure area should be closely monitored.

CHAPTER 11

FOUNDATION INSTRUMENTATION

11-01. General.

a. Purpose. Piezometers, both pressure cell and open tube types were installed to measure pore pressure in the embankment and spillway-powerhouse foundations.

b. Structures. Thirty-three pore pressure transducers were installed during construction; eighteen in the spillway foundation and fifteen in the powerhouse foundation. Each cell is located in a 4-inch diameter hole drilled 2 feet into bedrock. The tip of the pressure cell is in sand. Sand extends 18 inches above the bottom of the hole and is capped with a 3-inch seal of granular bentonite. For locations and details see Plates 156 and 157.

c. Rock. No observation devices were installed to monitor bedrock but deformeter bolts were installed in the excavated walls of the Spillway-Powerhouse. See paragraph 8-03.

d. Embankment. A total of 84 observation devices were installed to monitor the embankment. Seventeen pressure cells and seven open tube piezometers were installed; three in the impervious foundation, two in the impervious of the cutoff trench, fourteen in the pervious foundation and five in the impervious embankment. Also installed were: forty-eight alinement monuments, eight crest settlement monuments, two tiltmeters and two foundation settlement plates. Two strong-motion accelographs were also installed. One is at Sta. 55+00 near the dam crest on the upstream side of the roadway. The other is on the left abutment. See Plates 159 and 160.

e. Sterett Creek Embankment. A total of 32 observation devices were installed in the Sterett Creek embankment. Nineteen open tube piezometers were installed. Twelve had tips in the foundation clay and seven had tips in foundation sands and gravels. Eleven crest settlement monuments and two foundation settlement plates were also installed. See Plate 158.

f. Results. A continuous record of readings of all observation devices is maintained in the Foundations and Materials Branch of the Kansas City District Office.

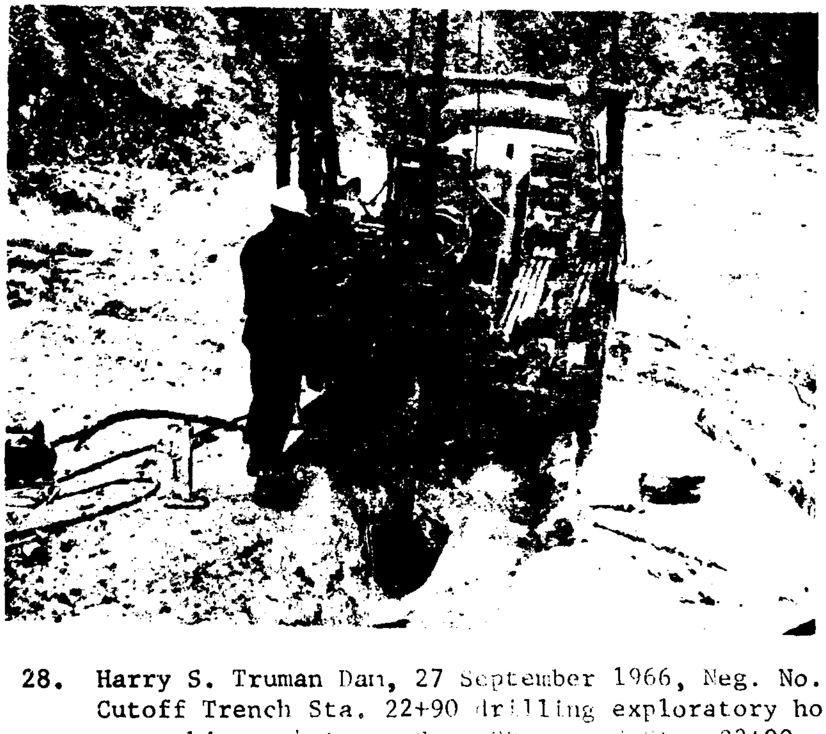
23. Harry S. Truman Dam, 30 August 1966, Neg. No. 82414. Cutoff Trench Sta. 61+60 looking downstream and upstation. Placing impervious fill.

24. Harry S. Truman Dam, 3 September 1966, Neg. No. 82419. Cutoff Trench Sta. 61+66. Looking upstation. Final cleanup.

25. Harry S. Truman Dam, 7 September 1966, Neg. No. 82421.
Cutoff Trench Sta. 23+00. Looking upstation.

26. Harry S. Truman Dam, 21 September 1966, Neg. No. 82491.
Cutoff Trench Sta. 23+00 downstream side of trench.

27. Harry S. Truman Dam, 22 September 1966, Neg. No. R-251-9. Cutoff Trench Sta. 22+40 looking upstation where N-S cave crosses trench.



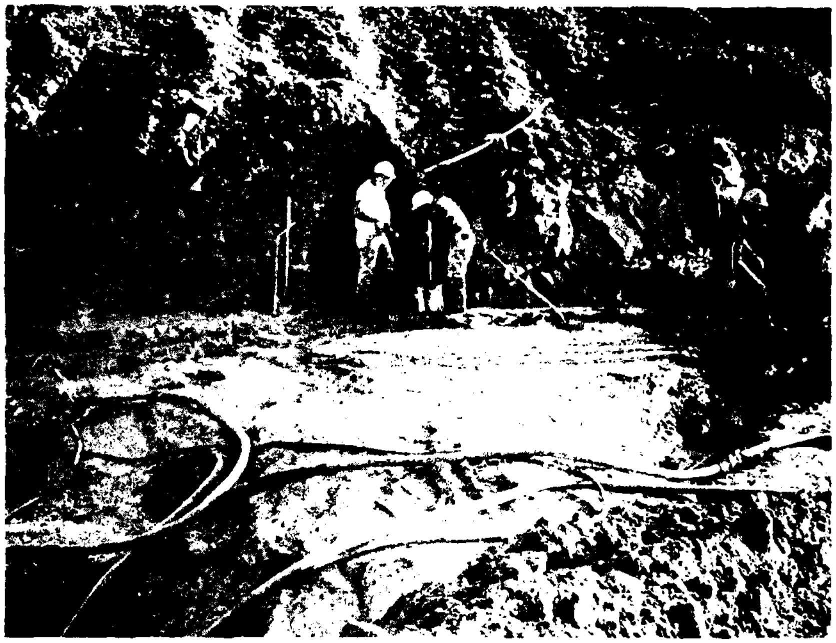
28. Harry S. Truman Dam, 27 September 1966, Neg. No. 82494. Cutoff Trench Sta. 22+90 drilling exploratory holes near chimney between Sta. 22+[illegible] and Sta. 23+00.

29. Harry S. Truman Dam, 10 November 1966, Neg. No. R-262-3. Cutoff Trench Sta. 23+33 to Sta. 23+50 looking upstation. Caves on downstream side.

30. Harry S. Truman Dam, 11 November 1966, Neg. No. R-262-6. Cutoff Trench Sta. 23+50 looking downstation. Sealed cave on downstream side.

31. Harry S. Truman Dam, 11 November 1966, Neg. No. R-262-8. Cutoff Trench Sta. 23+00 looking upstation. Caves on downstream side Sta. 23+40 to Sta. 23+80.

32. Harry S. Truman Dam, 11 November 1966, Neg. No. R-262-9. Cutoff Trench Sta. 23+00 looking upstation.

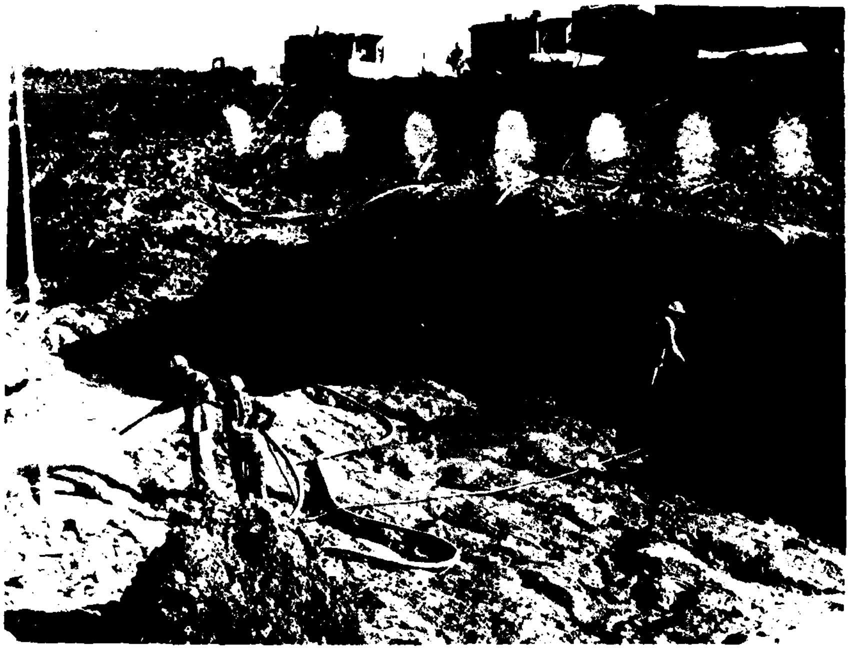


33. Harry S. Truman Dam, 14 November 1966, Neg. No. 82750. Cutoff Trench Sta. 25+00 downstream wall.

34. Harry S. Truman Dam, 18 November 1966, Neg. No. 82922. Cutoff Trench Sta. 26+00 to Sta. 27+00 looking upstation. Downstream side.

35. Harry S. Truman Dam, 19 November 1966, Neg. No. 82928. Cutoff Trench Sta. 27+25 to Sta. 27+75 Breccia Contact. Looking downstream and upstation.

36. Harry S. Truman Dam, 17 December 1966, Neg. No. 82893. Cutoff Trench Sta. 33+00 looking upstation.



37. Harry S. Truman Dam, 18 December 1966, Neg. No. 82895. Cutoff Trench Sta. 32+80 looking downstrem and upstation.

38. Harry S. Truman Dam, 19 December 1966, Neg. No. 82905, Cutoff Trench Sta. 34+00 to Sta. 34+70. Looking downstation on downstream side.

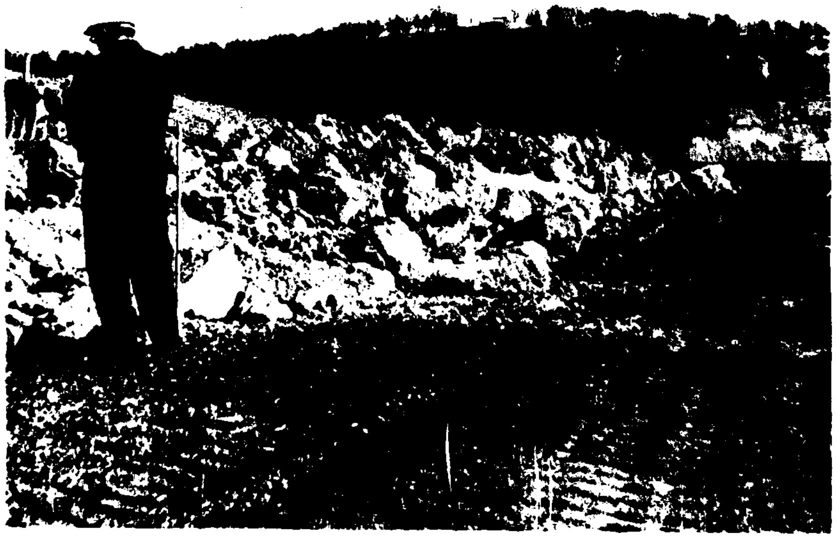
39. Harry S. Truman Dam, 19 December 1966, Neg. No. 82909, Cutoff Trench Sta. 35+50 looking downstream.

40. Harry S. Truman Dam, 19 December 1966, Neg. No. 82910. Cutoff Trench Sta. 36+26 looking downstream.

41. Harry S. Truman Dam, 28 October 1966, Neg. No. 82734. Spillway-Powerhouse excavation. Experimental blasting.

42. Harry S. Truman Dam, 28 October 1966, Neg. No. 82737. Spillway-Powerhouse excavation. Experimental blasting.

43. Harry S. Truman Dam, 7 November 1966, Neg. No. R-261-5. Test fill at Sta. 55+00 looking northeast.



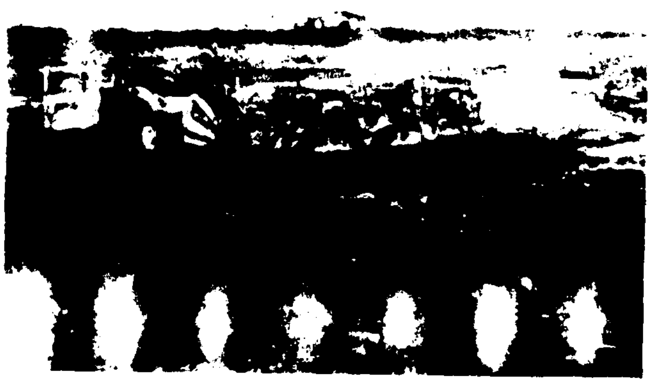
44. Harry S. Truman Dam, 8 November 1966, Neg. No. R-261-7. Test fill at Sta. 55+00 looking northeast.

45. Harry S. Truman Dam, 8 November 1966, Neg. No. R-261-8. Spillway-Powerhouse excavation. Experimental blasting.

46. Harry S. Truman Dam, 12 November 1966, Neg. No. 82742. Test fill at Sta. 55+00 looking northeast.

47. Harry S. Truman Dam, 11 April 1968, Neg. No. 85347. Spillway-Powerhouse rock excavation.

48. Harry S. Truman Dam, 11 January 1971, Neg. No. 965-16. Spillway-Powerhouse overburden excavatior Looking upstream. Building approach channel plug.



49. Harry S. Truman Dam, 16 June 1971, Neg. No. 965-123. Spillway-Powerhouse rock excavation. Upstream wall of right non-overflow bulkhead.

50. Harry S. Truman Dam, 5 May 1971, Neg. No. 965-122. Spillway-Powerhouse rock excavation just upstream of spillway piers. Looking upstream.

AD-A154 209 MULTIPLE-PURPOSE PROJECT OSAGE RIVER BASIN OSAGE RIVER MISSOURI HARRY S T.. (U) CORPS OF ENGINEERS KANSAS CITY MO KANSAS CITY DISTRICT R F GRIFFITH ET AL. 1984

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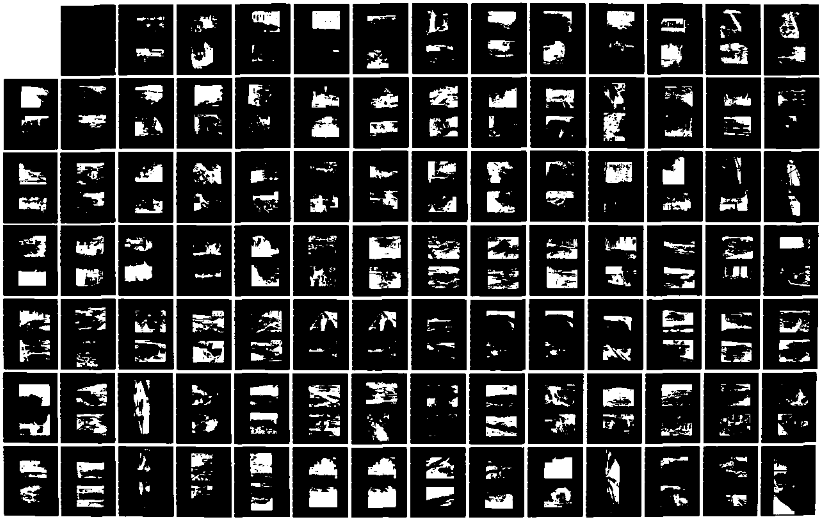

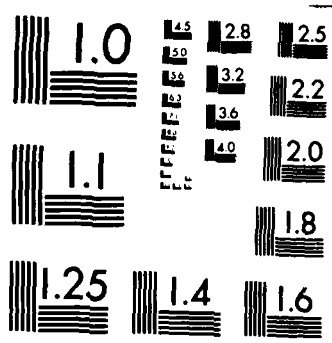
1.0
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51. Harry S. Truman Dam, 12 May 1971, Neg. No. 965-29. Spillway-Powerhouse rock excavation. Presplit face along Control Line B, Sta. 52+25 to Sta. 50+64 looking upstream Floor El. 660.

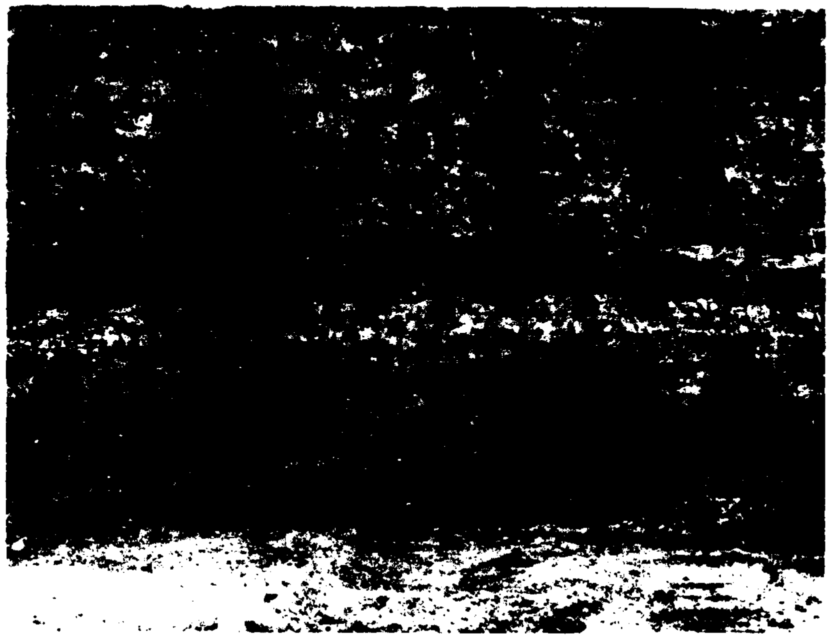
52. Harry S. Truman Dam, 12 May 1971, Neg. No. 965-99. Spillway-Powerhouse rock excavation. Presplit face along Line B, Sta. 50+64 to Sta. 52+25 looking downstream Floor El. 660.

53. Harry S. Truman Dam, 21 June 1971, Neg. No. 965-93. Spillway-Powerhouse rock excavation. Damaged corner at upstream of spillway training wall Sta. 50+74.29 looking northeast.

54. Harry S. Truman Dam, 23 June 1971, Neg. No. 965-83. Spillway-Powerhouse excavation. Rock bolting spillway training wall. Looking downstream.

55. Harry S. Truman Dam, 3 September 1971, Neg. No. 965-115. Spillway-Powerhouse excavation. Joy Ram Drill installing rock bolts in tailrace training wall Sta. 51+90 Control Line B.

56. Harry S. Truman Dam, 15 September 1971, Neg. No. 965-111. Spillway-Powerhouse. Rock bolts in Spillway training wall Sta. 50+74 Control Line C.



57. Harry S. Truman Dam, 15 September 1971, Neg. No. 965-21. Spillway-Powerhouse. Rock bolts in Spillway training wall Sta. 52+80 Control Line C.

58. Harry S. Truman Dam, 15 September 1971, Neg. No. 965-24. Spillway-Powerhouse. Rock bolts in Spillway training wall Sta. 51+10 to Sta. 53+00 Control Line C.

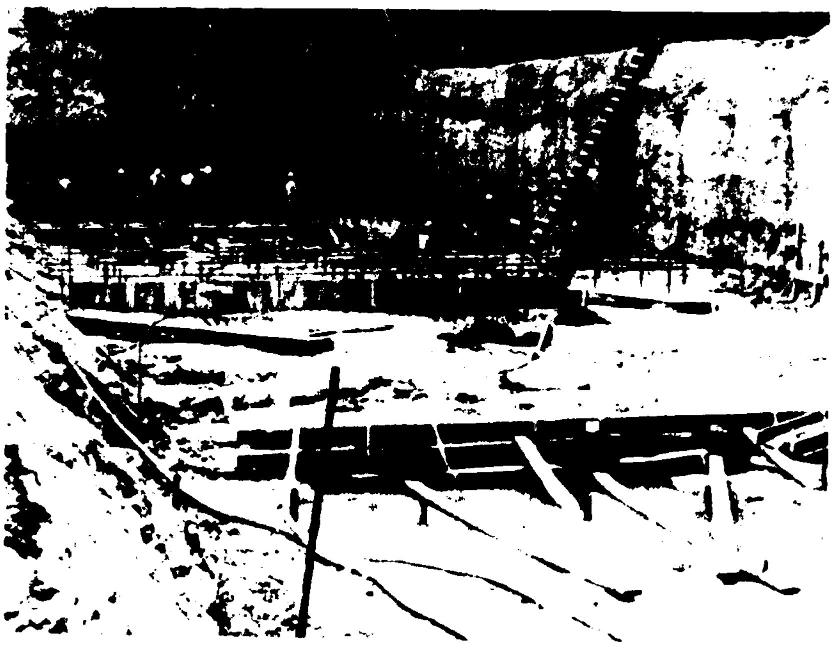
59. Harry S. Truman Dam, 23 September 1971, Neg. No. 965-118. Spillway-Powerhouse. Rock excavation of outlet channel Sta. 56+50 Control Line A. Looking upstream toward tailrace training wall.

60. Harry S. Truman Dam, 29 September 1971, Neg. No. 965-12. Spillway-Powerhouse. Foundation and upstream wall or Monolith 1.

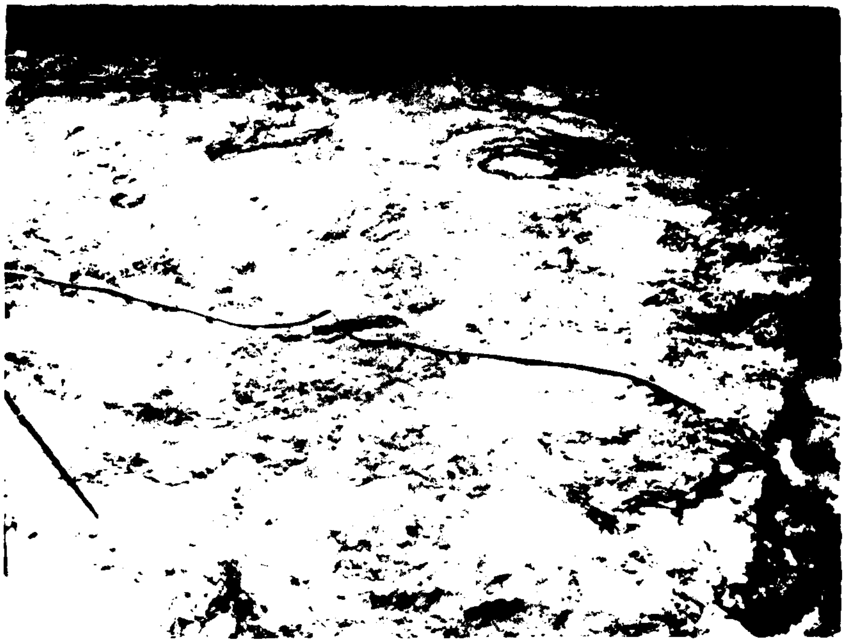
61. Harry S. Truman Dam, 19 October 1971, Neg. No. 965-28. Spillway-Powerhouse. Placing first concrete in sump of left wall of erection bay.

62. Harry S. Truman Dam, 22 October 1971, Neg. No. 965-81 Spillway-Powerhouse. Foundaton vicinity of divider wall. Looking east toward spillway training wall.

63. Harry S. Truman Dam, 22 October 1971, Neg. No. 965-88. Spillway-Powerhouse. Foundation Monolith 8. El. 583. Looking Northeast toward spillway training wall.



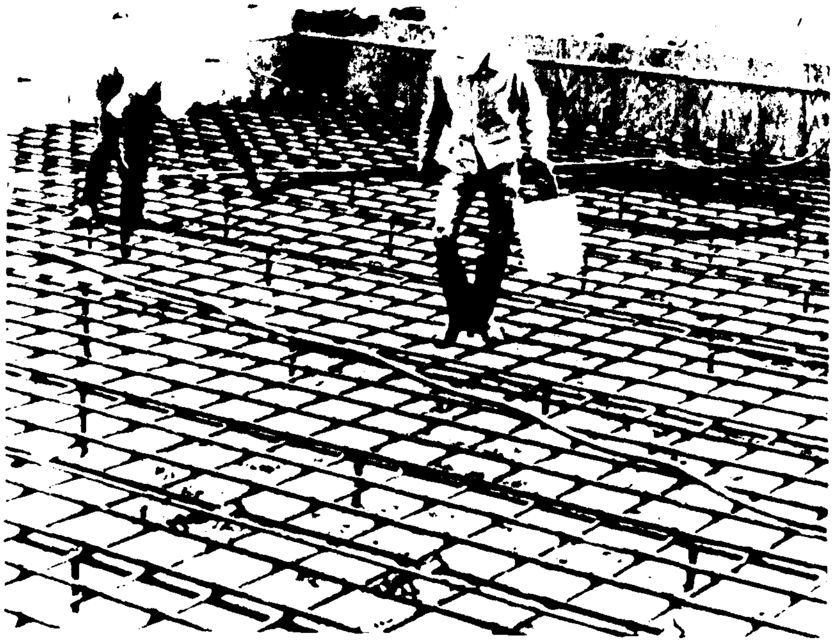
64. Harry S. Truman Dam, 22 October 1971, Neg. No. 965-86. Spillway-Powerhouse. Foundation and upstream wall of Monoliths 6 and 7. El. 583. Looking southwest toward right wall of powerhouse.



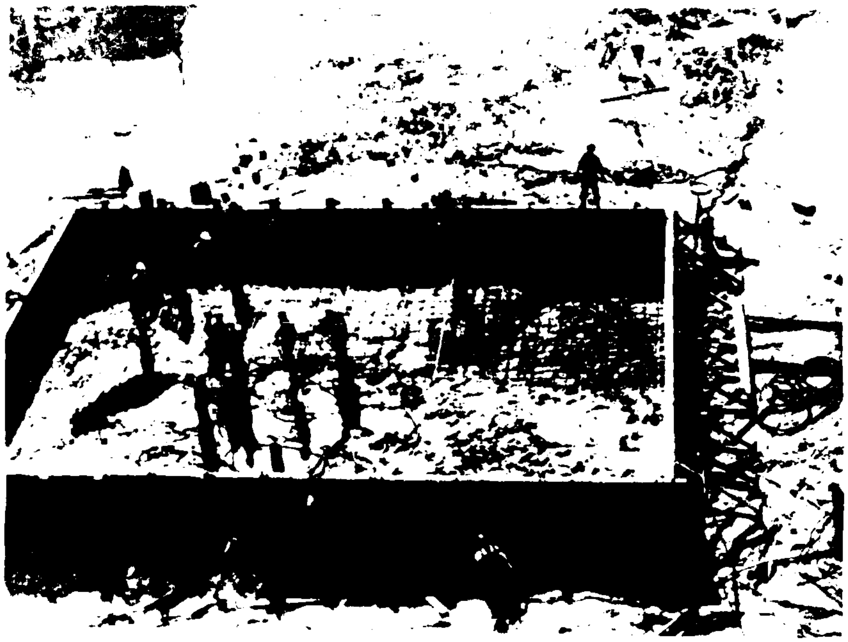
65 Harry S. Truman Dam, 1 November 1971, Neg. No. 965-22. Spillway-Powerhouse. Foundation downstream wall and right wall of Monolith 1. Looking downstream.

66. Harry S. Truman Dam, 1 November 1971, Neg. No. 965-78. Spillway-Powerhouse. Foundation upstream wall of Monoliths 6 and 7 and right wall of Powerhouse. Looking west.

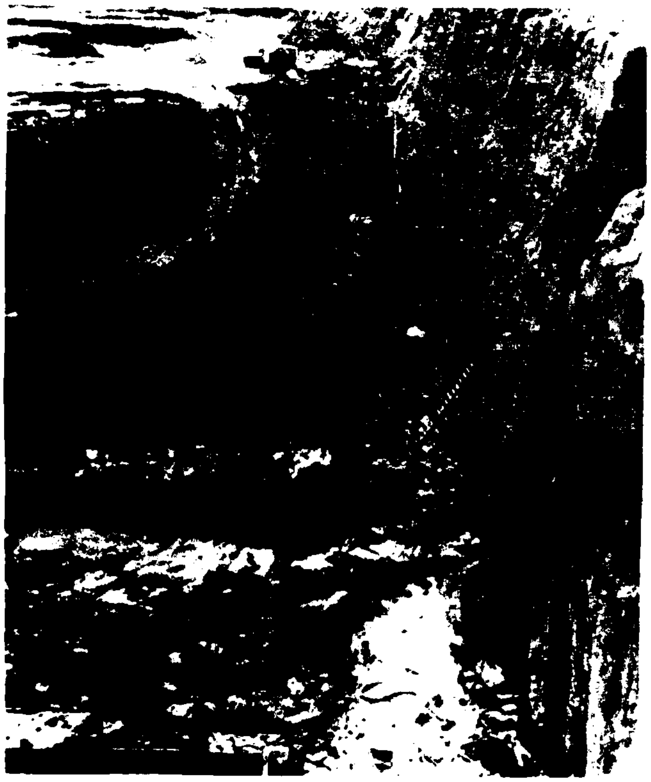
67. Harry S. Truman Dam, 1 November 1971, Neg. No. 965-89. Spillway-Powerhouse. Foundation of Monoliths 6 and 7 on El. 589 bench. Looking west.



68. Harry S. Truman Dam, 2 November 1971, Neg. No. 965-87. Spillway-Powerhouse. Foundation Monolith 6, concrete pour 3h. Looking west.

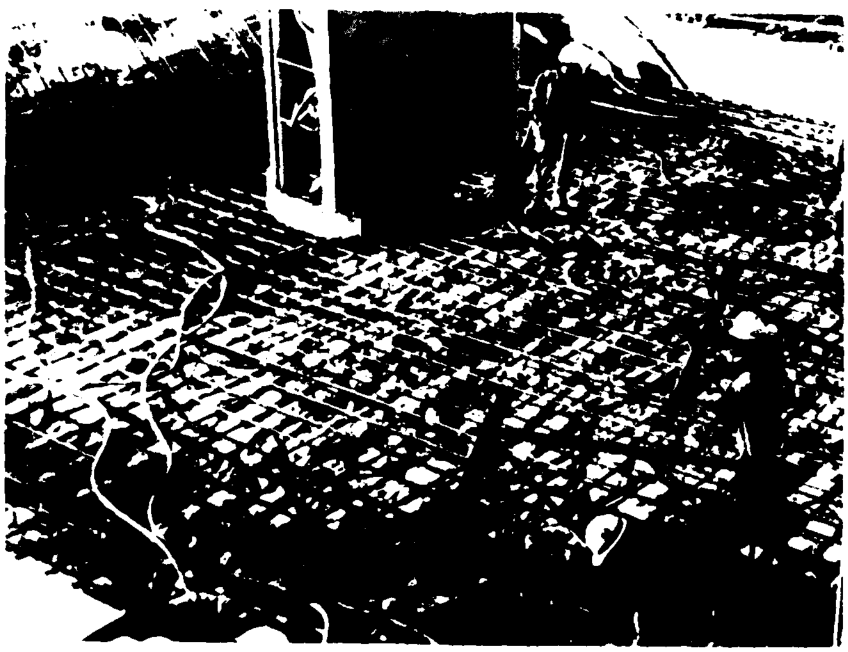


69. Harry S. Truman Dam, 2 November 1971, Neg. No. 965-84. Spillway-Powerhouse. Foundation Monolith 6, concrete pour 3h. Looking downstream from Sta. 49+60, Control Line C.



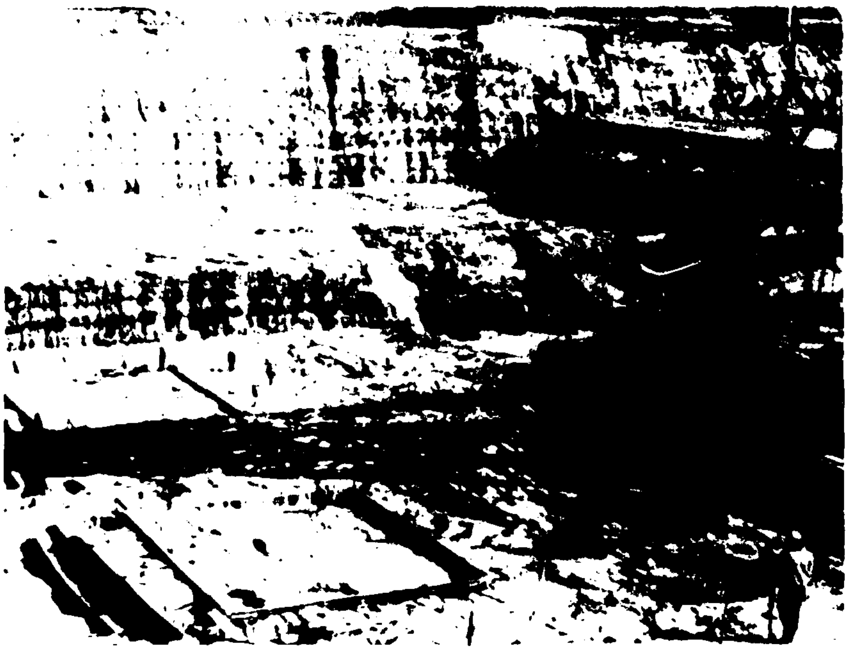
70. Harry S. Truman, 2 November 1971, Neg. No. 965-2. Spillway-Powerhouse. Tailrace training wall Looking downstream from Sta. 50+00, Control Line C. Drilling drain holes.

71. Harry S. Truman Dan, 3 November 1971, Neg. No. 965-54. Spillway-Powerhouse. Foundation of upstream part of Monoliths 7 and 8 on El. 583 bench. Looking downstream and left toward divider wall. View from Sta. 49+60, Control Line C.



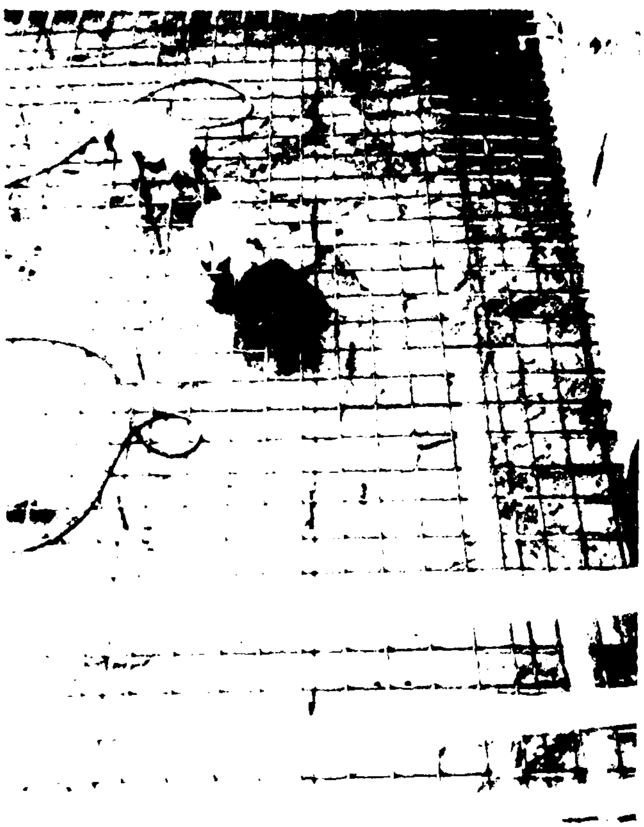
72. Harry S. Truman Dam, 3 November 1971, Neg. No. 965-45. Spillway-Powerhouse. Foundation Monolith 6 El. 583. Placing concrete pour 1b. Looking downstream and right.

73. Harry S. Truman Dam, 3 November 1971, Neg. No. 965-48. Spillway-Powerhouse. Foundation Monoliths 6, 7, & 8 El. 583. Placing concrete pour 3h. Looking left from Sta. 49+60 Control Line C.



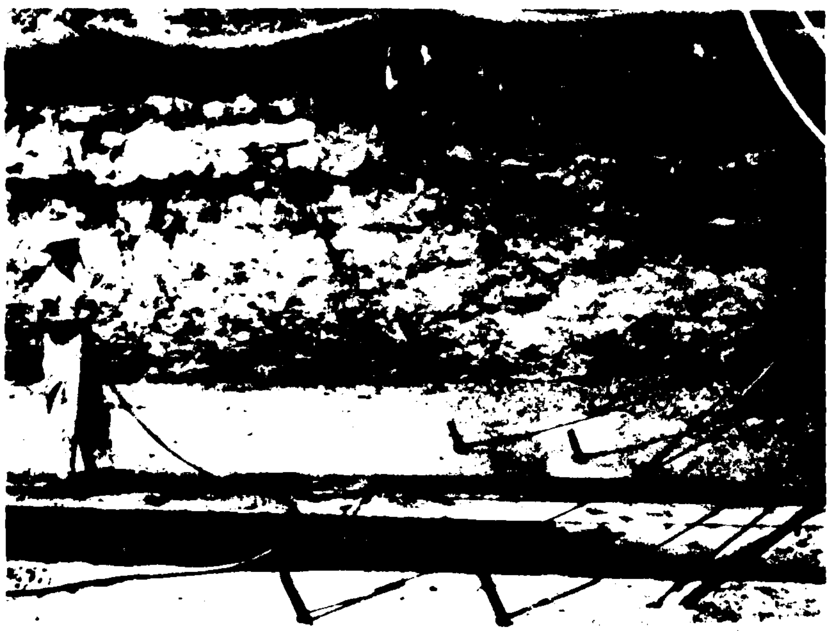
74. Harry S. Truman Dam, 3 November 1971, Neg. No. 965-47. Spillway-Powerhouse. Foundation Monoliths 6, 7 and 8. Placing concrete pour 3h. Looking left from Sta. 49+60 Control Line C.

101. Harry S. Truman Dam, 20 April 1972, Neg. No. 965-56. Spillway-Powerhouse. Monolith 11, right wall. Ready for concrete pour 4.

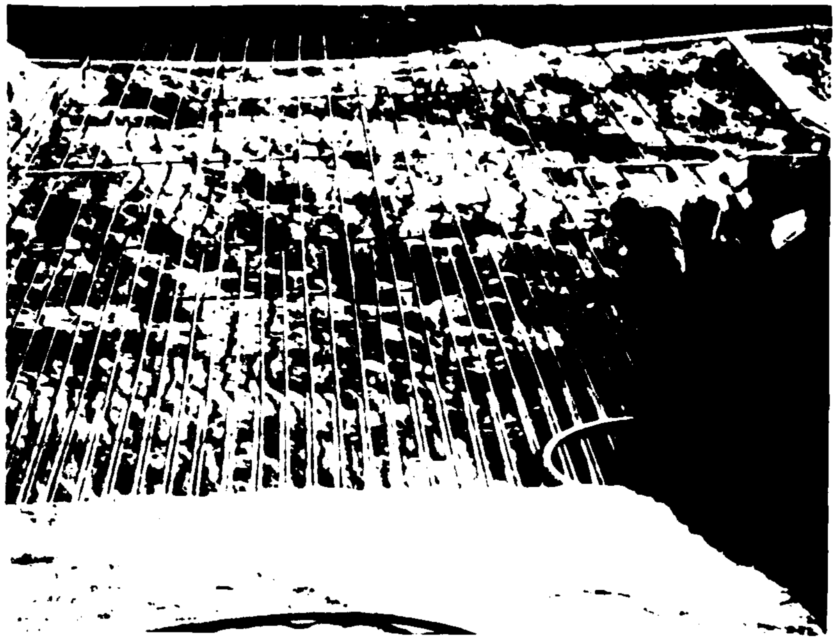


102. Harry S. Truman Dam, 27 April 1972, Neg. No. 965-43. Spillway-Powerhouse. Monolith 6. Ready for concrete pour 3n.

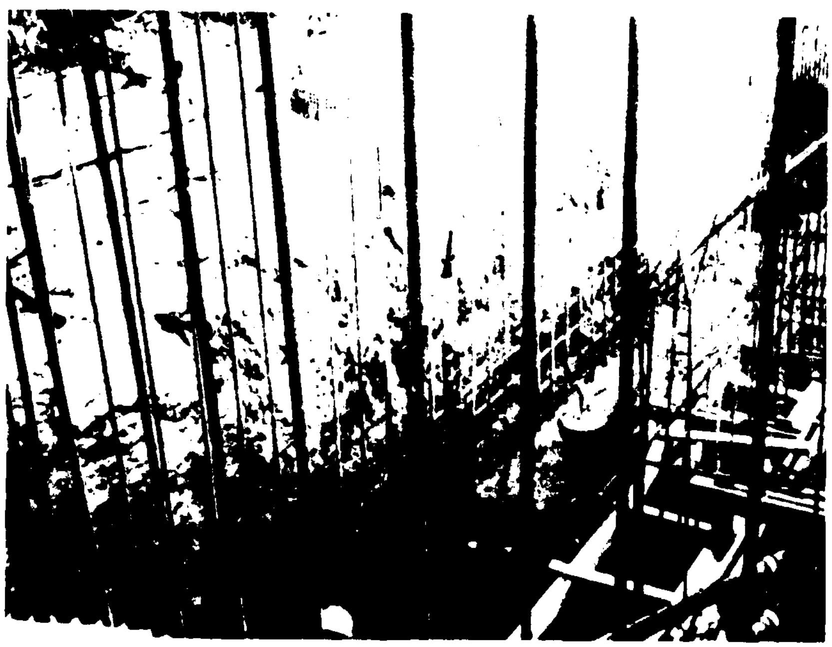
99. Harry S. Truman Dam, 18 April 1972, Neg. No. 965-42. Spillway-Powerhouse. Monolith 3. Ready for concrete pour 4a.



100. Harry S. Truman Dam, 20 April 1972, Neg. No. 965-73. Spillway-Powerhouse. Monolith 11, left wall. Ready for concrete pour 4.



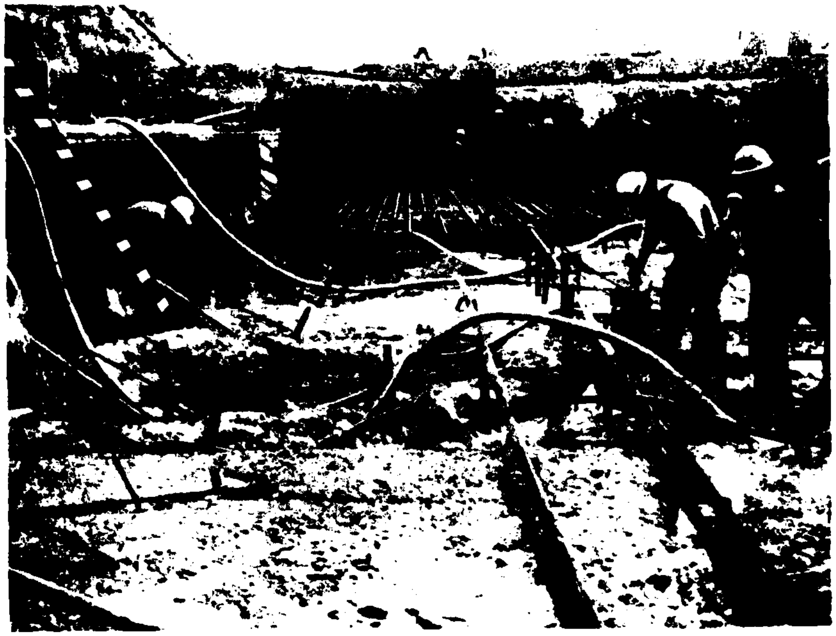
97. Harry S. Truman Dam, 16 March 1972, Neg. No. 965-53. Spillway-Powerhouse. Divider wall Monolith DW-1. Ready for concrete left 3.



98. Harry S. Truman Dam, 18 April 1972, Neg. No. 965-44. Spillway-Powerhouse. Monolith 8. Ready for concrete pour 2c.

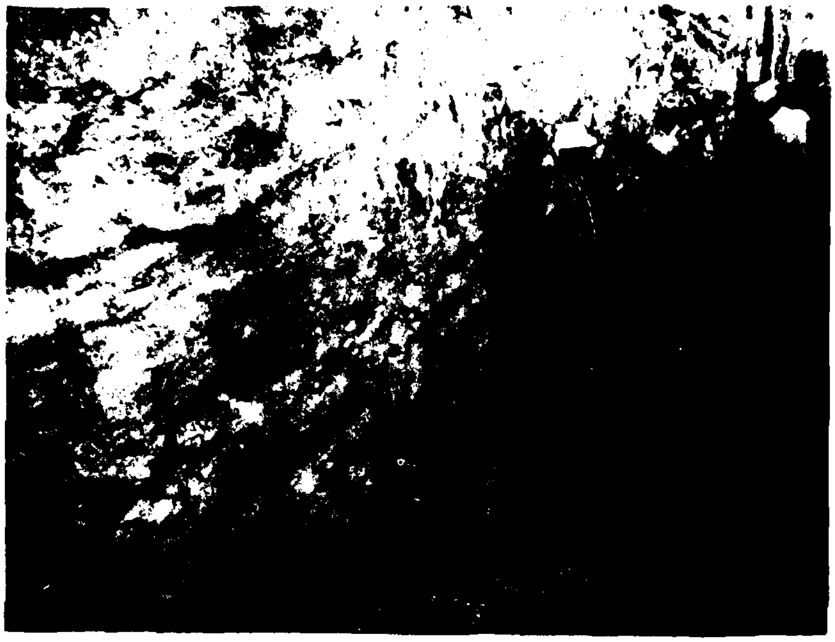
95. Harry S. Truman Dam,
13 April 1972,
Neg. No. 965-103.
Spillway-Powerhouse.
Downstream wall of
Monolith 10 open
joints. Limestone
blocks to be
removed.

96. Harry S. Truman Dam,
14 April 1972,
Neg. No. 965-76.
Spillway-Powerhouse.
Downstream wall of
Monolith 10.

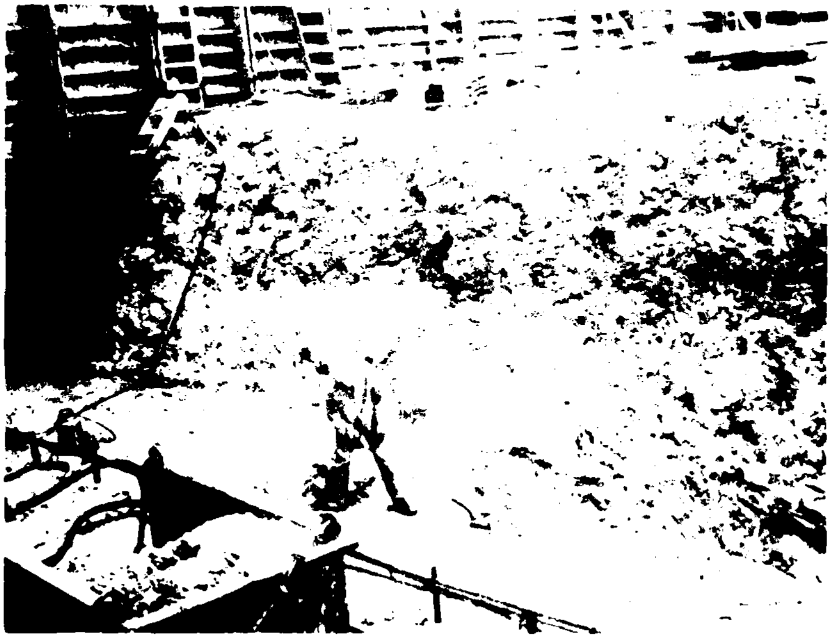


93. Harry S. Truman Dam, 11 April 1972, Neg. No. 965-104. Spillway-Powerhouse. Monolith 11 3rd lift of concrete looking downstream.

94. Harry S. Truman Dam, 13 April 1972, Neg. No. 965-96. Spillway-Powerhouse. Foundation and downstream wall of Monolith 12.

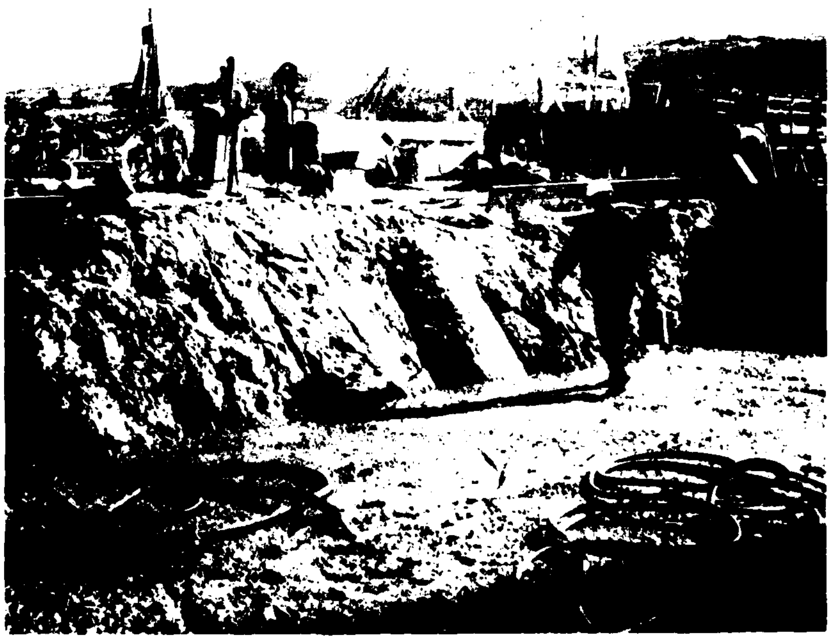


91. Harry S. Truman Dam, 5 April 1972, Neg. No. 965-30. Spillway-Powerhouse. Spillway approach wall looking downstream.

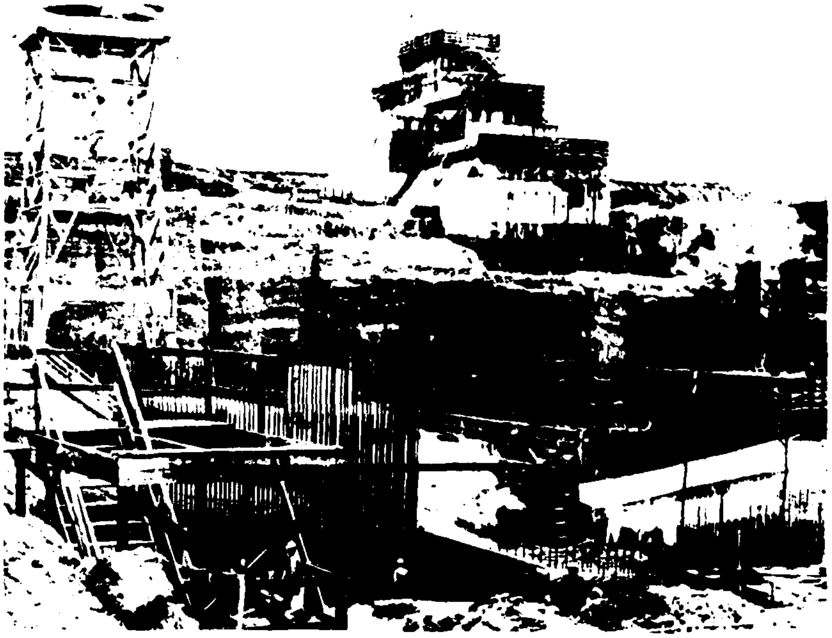


92. Harry S. Truman Dam, 7 April 1972, Neg. No. 965-105. Spillway-Powerhouse. Foundation and downstream wall of Monolith 10.

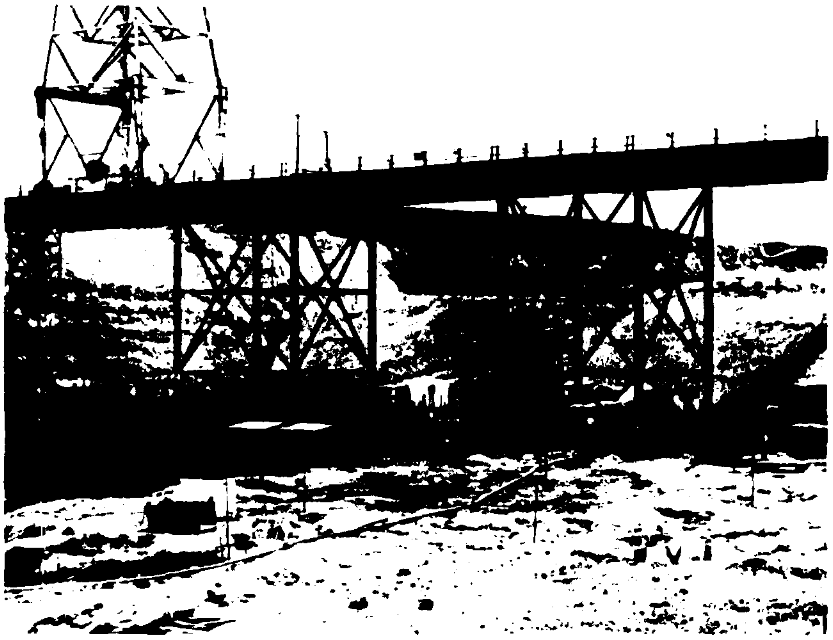
89. Harry S. Truman Dam, 3 April 1972, Neg. No. 965-32. Spillway-Powerhouse. View from Whirly Crane Bridge Looking downstream and to right.



90. Harry S. Truman Dam, 5 April 1972, Neg. No. 965-32. Spillway-Powerhouse. Downstream wall Monolith 11.

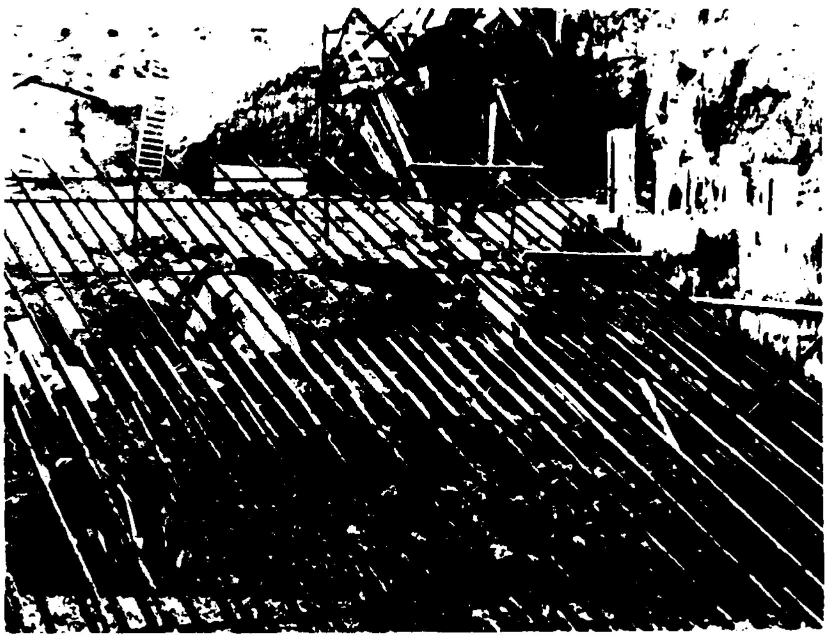


87. Harry S. Truman Dam, 3 April 1972, Neg. No. 965-59.
Spillway-Powerhouse. Looking upstream and to right.



88. Harry S. Truman Dam, 3 April 1972, Neg. No. 965-67.
Spillway-Powerhouse. Looking upstream and to right.

85. Harry S. Truman Dam, 3 April 1972, Neg. No. 965-60. Spillway-Powerhouse. Foundation of divider wall Monolith DW-3, slope 1V on 2.5H. Looking left.



86. Harry S. Truman Dam, 3 April 1972, Neg. No. 965-66. Spillway-Powerhouse. Foundaton of divider wall Monolith DW-3, slope 1V on 2.5H. Looking left.

83. Harry S. Truman Dam, 31 March 1972, Neg. No. 965-113. Spillway-Powerhouse. Foundation Monolith 11.

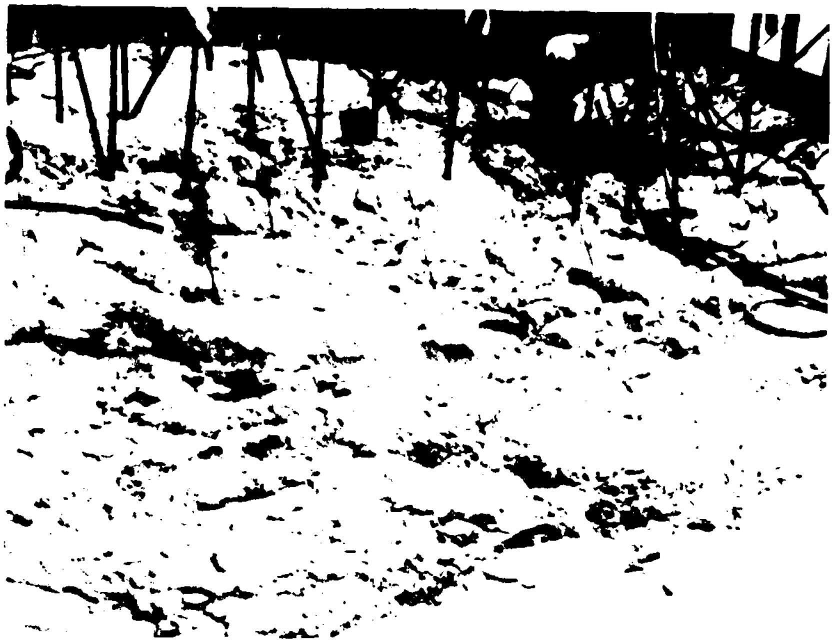
84. Harry S. Truman Dam, 31 March 1972, Neg. No. 965-14. Spillway-Powerhouse. Foundation and downstream wall of Monolith 11.

81. Harry S. Truman Dam, 8 December 1971, Neg. No. 965-124. Spillway-Powerhouse concrete. Looking upstream and toward right wall.

82. Harry S. Truman Dam, 23 March 1972, Neg. No. R-965-116 Spillway-Powerhouse. Rock Surface to receive backfill concrete along downstream wall of Monolith 3 looking right.

79. Harry S. Truman Dam, 17 November 1971, Neg. No. 965-91.
Spillway-Powerhouse. Foundation Monolith 6 and 7.
Looking at right wall of Powerhouse, El 606 at Sta. 51+70,
Control Line C.

80. Harry S. Truman Dam, 8 December 1971, Neg. No. 965-18.
Spillway-Powerhouse. Right wall of erection bay.
Looking upstream Sta. 50+54 to Sta. 49+80,
Control Line C.

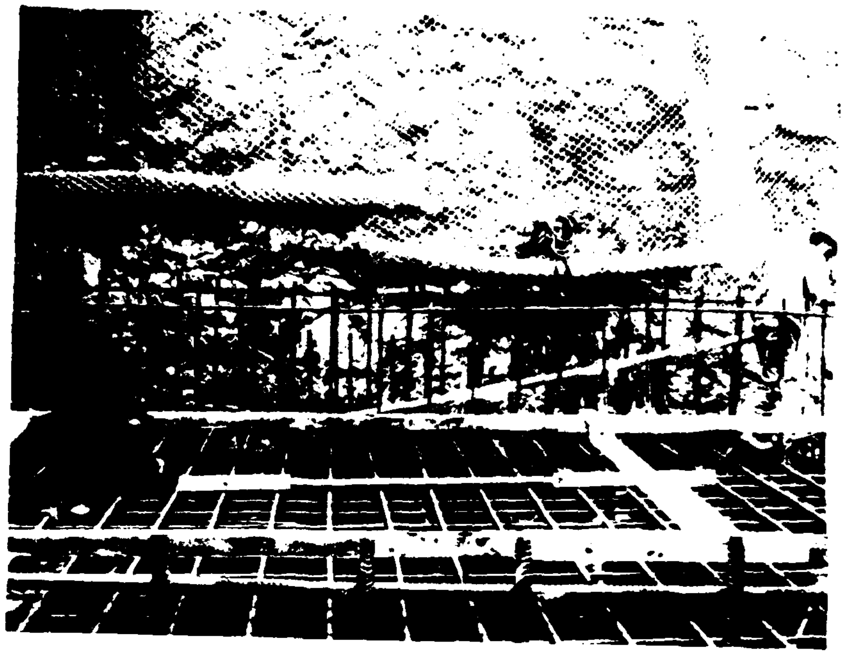


77. Harry S. Truman Dam, 9 November 1971, Neg. No. 965-71. Spillway-Powerhouse. Foundation Monolith 3 looking upstream.

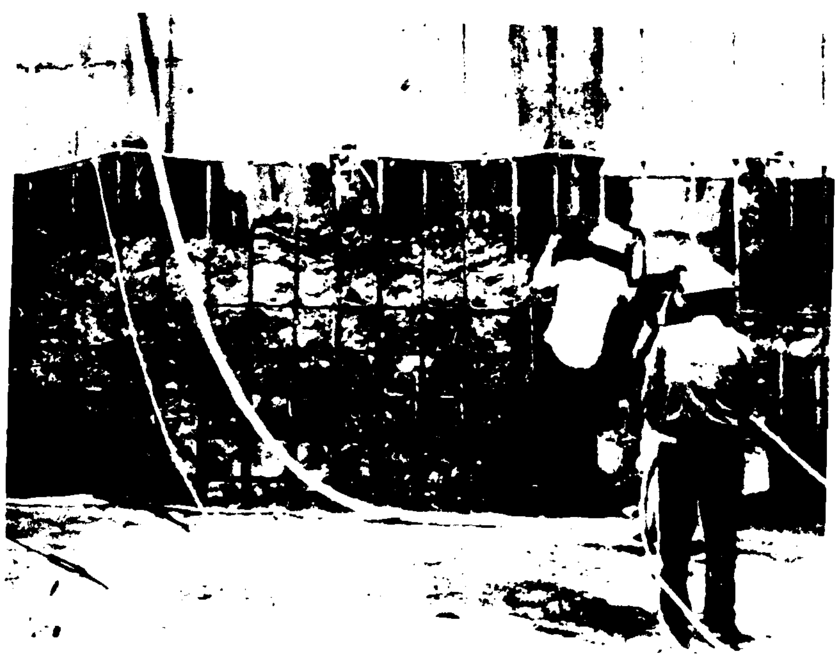
78. Harry S. Truman Dam, 17 November 1971, Neg. No. 965-92. Spillway-Powerhouse. Foundation Monoliths 6 and 7. Looking at right wall of Powerhouse El. 606 at Sta. 51+70, Control Line C.

75. Harry S. Truman Dam, 4 November 1971, Neg. No. 965-3. Spillway-Powerhouse. Foundation Monolith 1 looking upstream.

76. Harry S. Truman Dam, 9 November 1971, Neg. No. 965-85. Spillway-Powerhouse. Foundation Monolith 3 looking upstream.

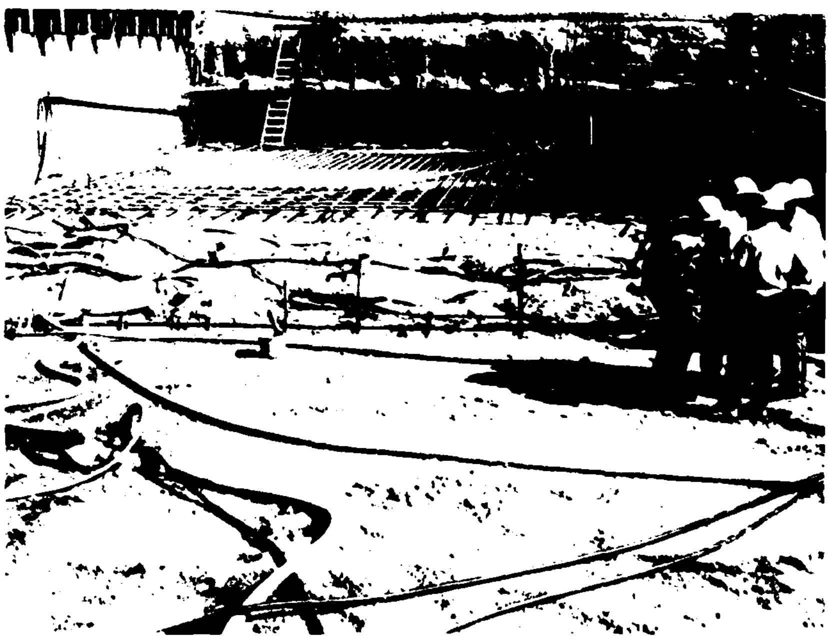


103. Harry S. Truman Dam, 28 April 1972, Neg. No. 965-50. Spillway-Powerhouse. Monolith 6, right wall. Ready for concrete pour 6a.



104. Harry S. Truman Dam, 28 April 1972, Neg. No. 965-35. Spillway-Powerhouse. Monolith 5, right wall. Ready for concrete pour 3c.

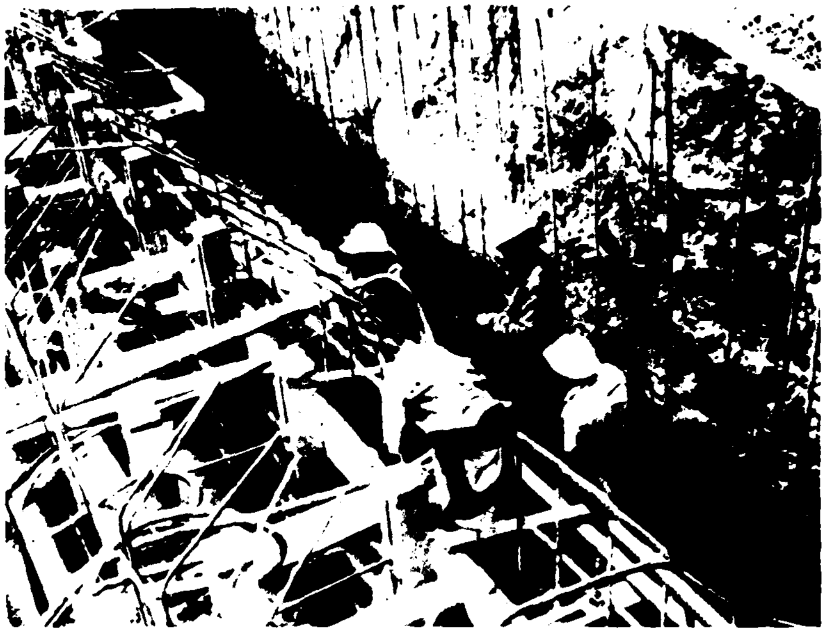
105. Harry S. Truman Dam, 1 May 1972, Neg. No. 965-34. Spillway-Powerhouse. Monolith 10. Foundation surface bedrock Unit 15-C.



106. Harry S. Truman Dam, 2 May 1972, Neg. No. 965-33. Spillway-Powerhouse. Downstream wall of Monolith 10. Ready for concrete left 3.

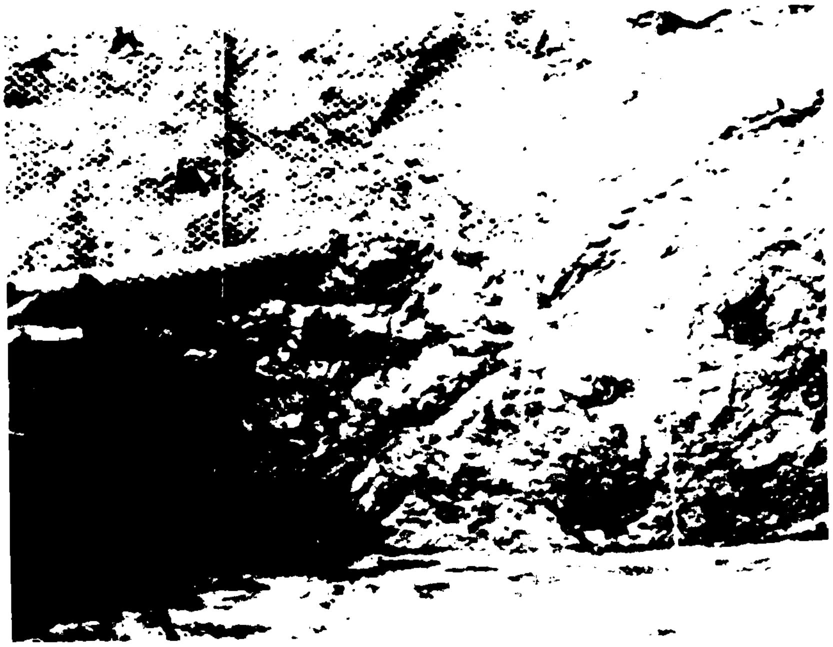
107. Harry S. Truman Dam, 3 May 1972, Neg. No. 965-40. Spillway-Powerhouse. Upstream wall of Monolith 9. 1V on 0.75H, slope on left. Ready for concrete lift 3.

108. Harry S. Truman Dam, 3 May 1972, Neg. No. 965-36. Spillway-Powerhouse. Upstream wall of Monolith 9. Ready for concrete left 3.



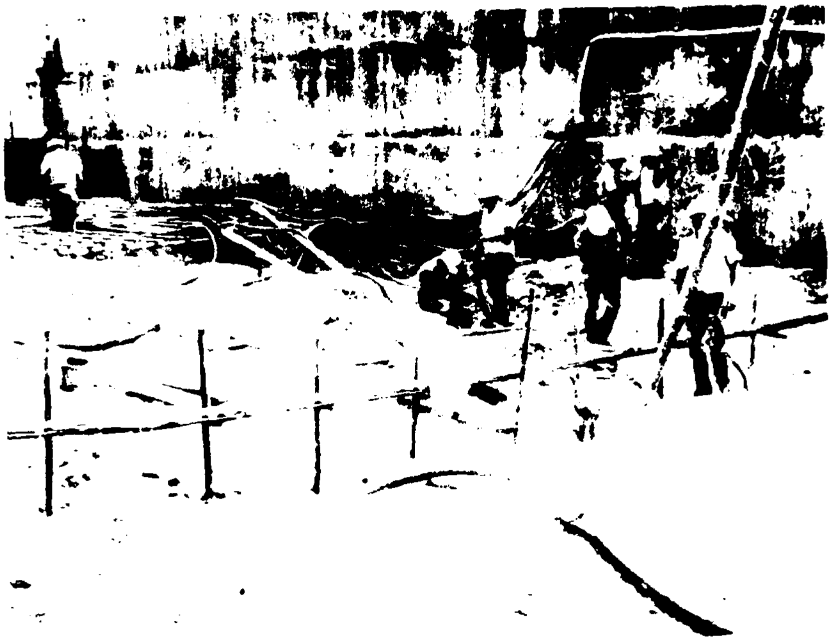
109. Harry S. Truman Dam, 3 May 1972, Neg. No. 965-37.
Spillway-Powerhouse. Upstream wall of Monolith 6.
Ready for concrete pour 4d.

110. Harry S. Truman Dam, 5 May 1972, Neg. No. 965-38.
Spillway-Powerhouse. Downstream wall Monolith 3.
Ready for concrete pour 2.

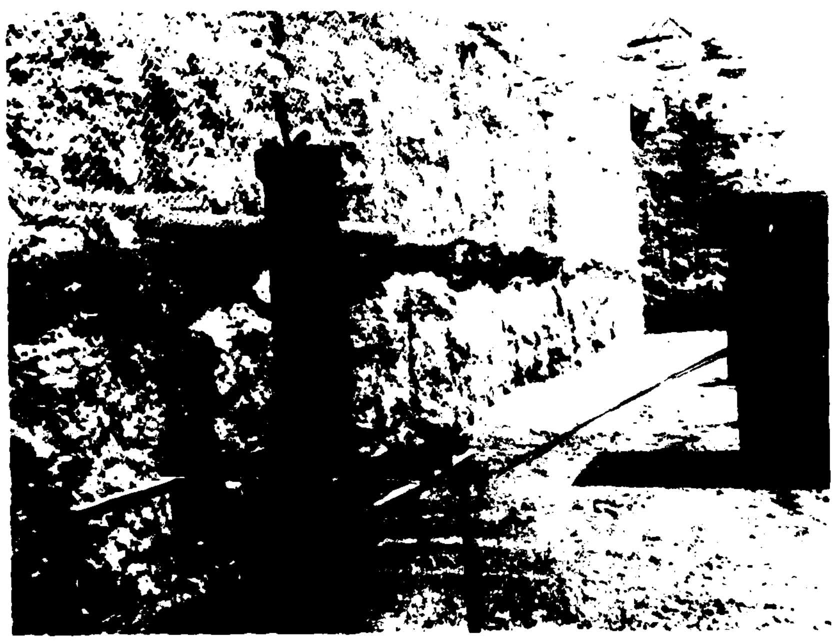


111. Harry S. Truman Dam, 24 May 1972, Neg. No. 965-39.
Spillway-Powerhouse. Right wall Monolith 13.
Ready for concrete lift 5.

112. Harry S. Truman Dam, 7 June 1972, Neg. No. 965-71.
Spillway-Powerhouse. Foundation Monolith 12.
Looking downstream. Ready for concrete lift 3.



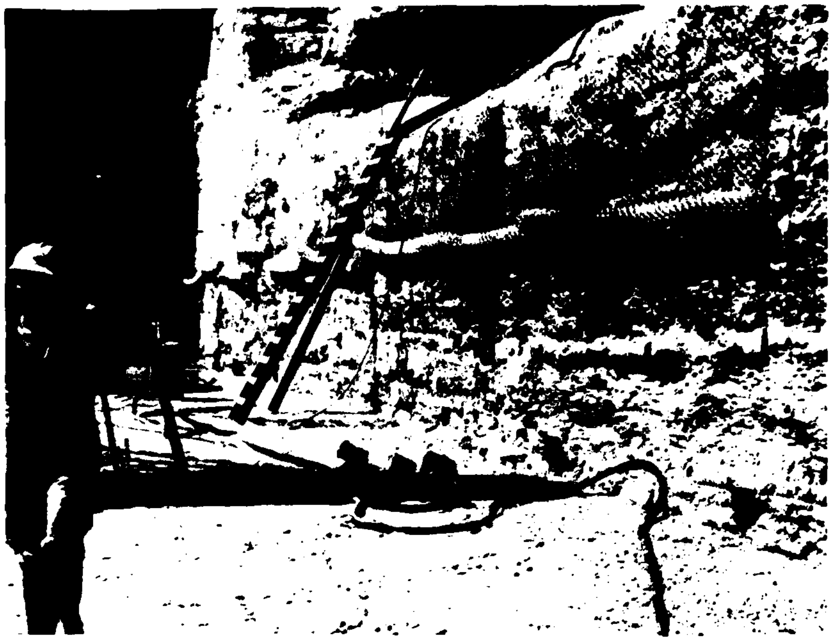
113. Harry S. Truman Dam, 7 June 1972, Neg. No. 965-62.
Spillway-Powerhouse. Foundation Monolith 12.
Looking downstream. Ready for concrete lift 3.



114. Harry S. Truman Dam, 9 June 1972, Neg. No. 965-5.
Spillway-Powerhouse. Left wall of Monolith 13.
Ready for concrete lift 7.

115. Harry S. Truman Dam, 20 June 1972, Neg. No. 965-58. Spillway-Powerhouse. Divider wall Monolith DW-4. Looking downstream.

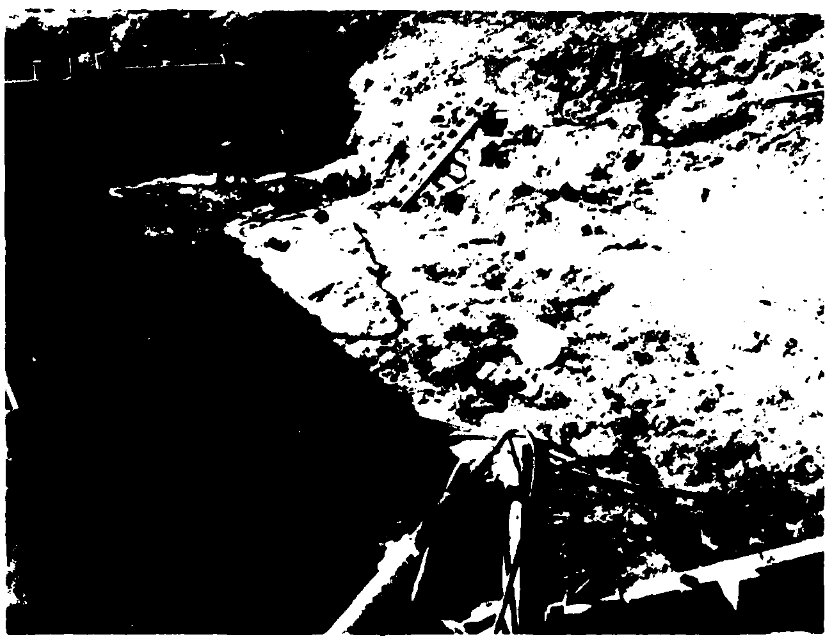
116. Harry S. Truman Dam, 26 June 1972, Neg. No. 965-57. Spillway-Powerhouse. Downstream wall and left wall of Monolith 13. Ready for concrete lift 8.



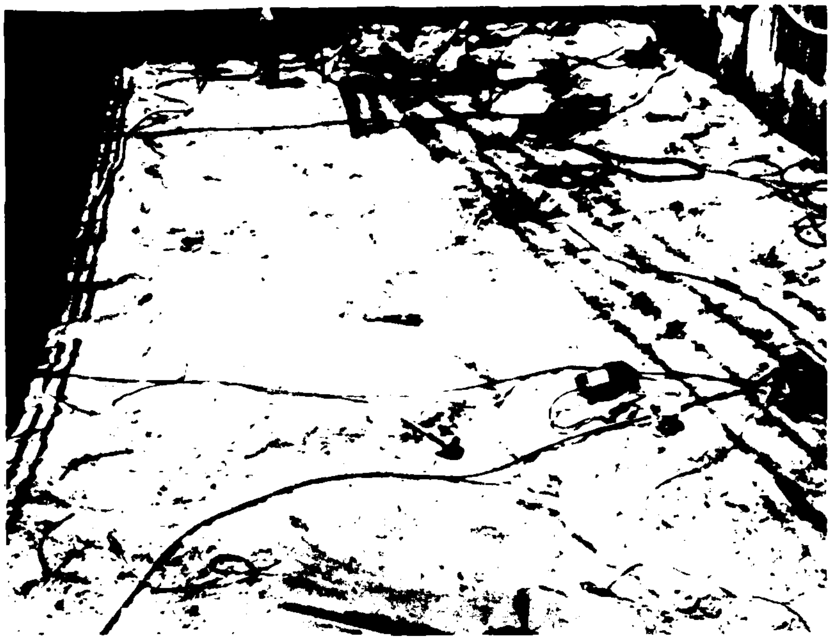
117. Harry S. Truman Dam, 26 June 1972, Neg. No. 965-4. Spillway-Powerhouse. Left wall of Monolith 13. Looking upstream. Ready for concrete lift 8.

118. Harry S. Truman Dam, 7 July 1972, Neg. No. 965-61. Spillway-Powerhouse. Downstream wall of Monolith 9. IV on 0.7H slope.

119. Harry S. Truman Dam, 7 July 1972, Neg. No. 965-68. Spillway-Powerhouse. Foundation Monolith 9. Looking downstream.

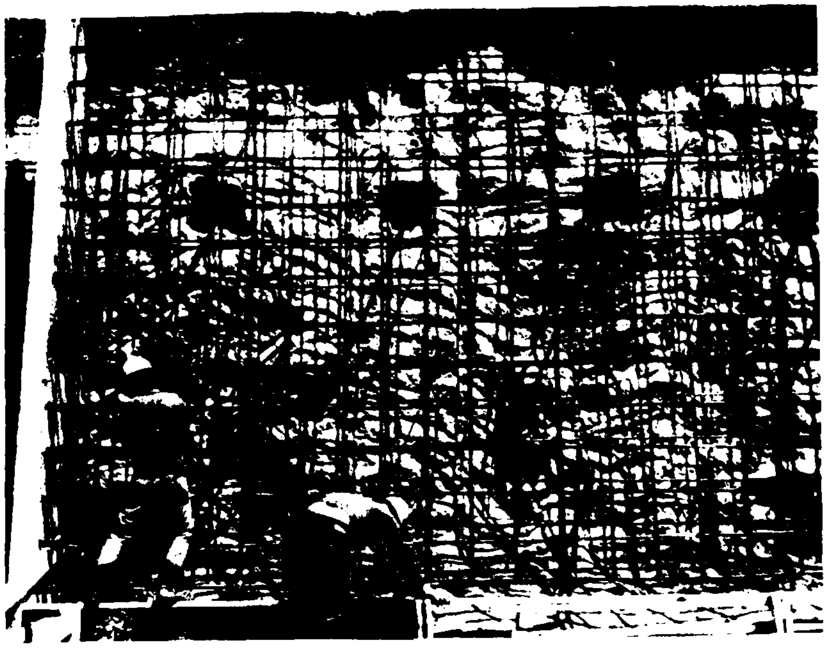


120. Harry S. Truman Dam, 14 July 1972, Neg. No. 965-9. Spillway-Powerhouse. Foundation Monolith 18. Looking upstream.

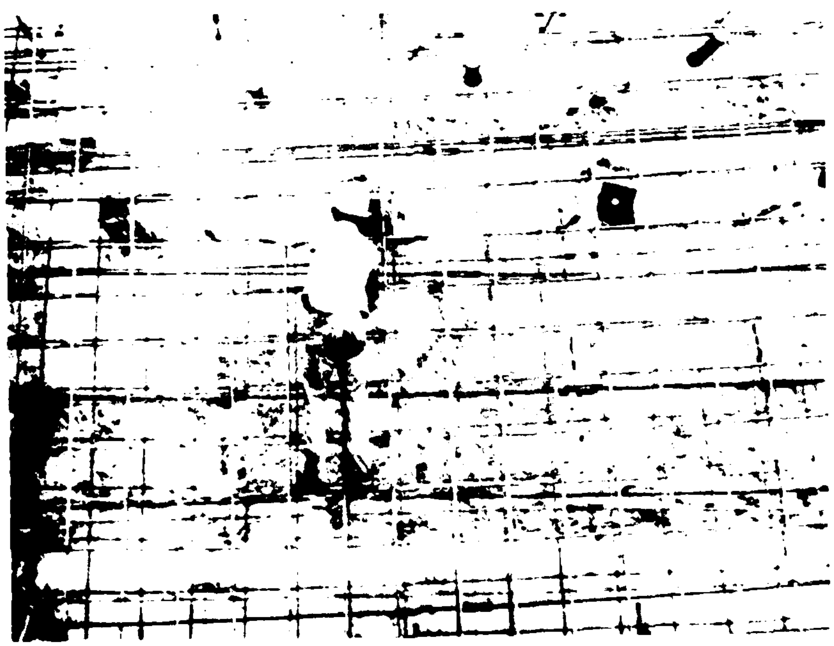


121. Harry S. Truman Dam, 31 July 1972, Neg. No. 965-72.
Spillway-Powerhouse. Foundation Monolith 14.
Looking upstream.

122. Harry S. Truman Dam, 31 July 1972, Neg. No. 965-77.
Spillway-Powerhouse. Foundation Monolith 14.
Looking upstream.

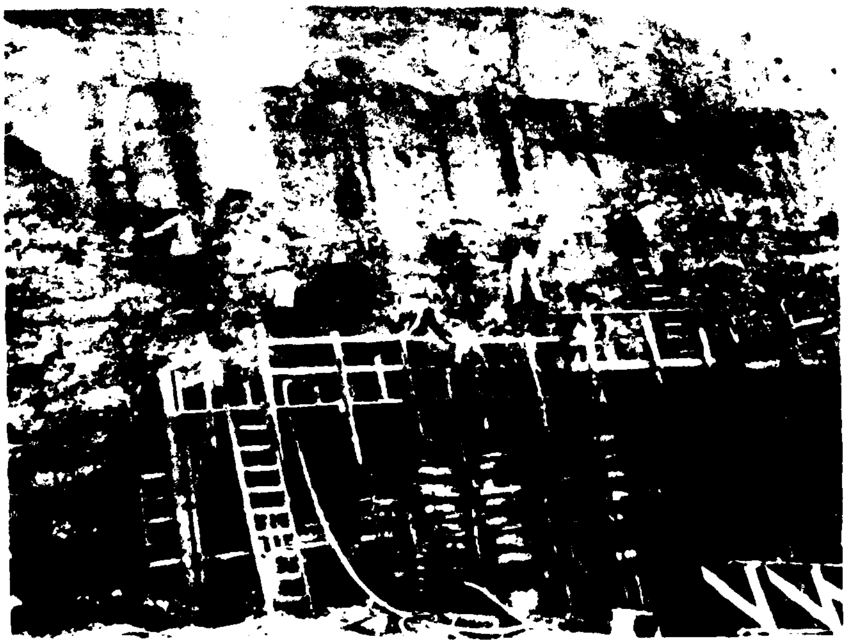


123. Harry S. Truman Dam, 5 September 1972, Neg. No. 965-70. Spillway-Powerhouse. Spillway training wall Monolith SW-4.



124. Harry S. Truman Dam, 5 September 1972, Neg. No. 965-69. Spillway-Powerhouse. Spillway training wall Monolith SW-5.

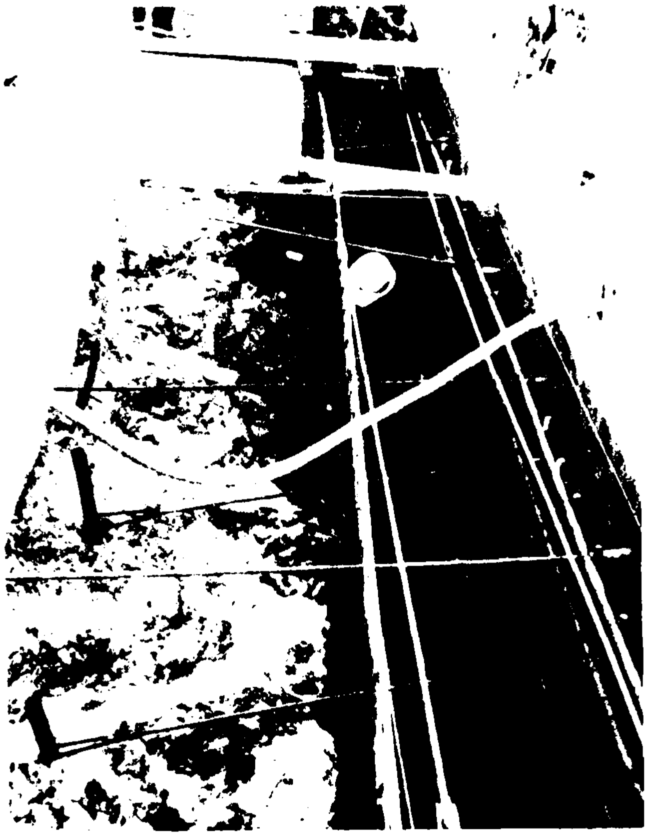
125. Harry S. Truman Dam, 11 September 1972, Neg. No. 965-125, Spillway-Powerhouse. Downstream wall of Monolith 9. Ready for concrete lift 4.



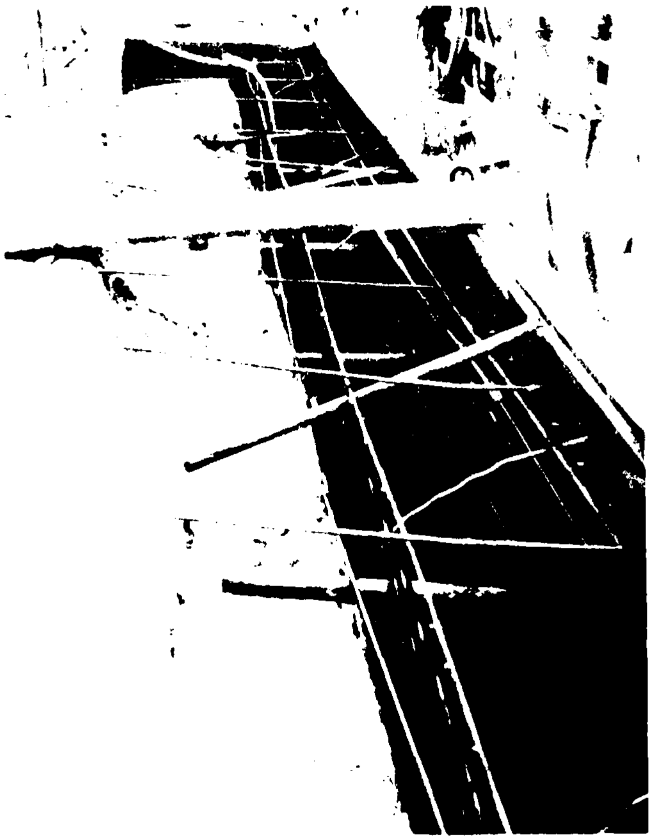
126. Harry S. Truman Dam, 6 November 1972, Neg. No. 965-26. Spillway-Powerhouse. Tailrace training wall Monolith TW-6. Scaling loose rock. Ready for concrete lift 7.

127. Harry S. Truman Dam, 6 November 1972, Neg. No. 965-26. Spillway-Powerhouse. Tailrace t aining wall Monolith TW-4. Looking upstream. Ready for concrete lift 7.

128. Harry S. Truman Dam, 7 November 1972, Neg. No. 965-107. Spillway-Powerhouse. Tailrace training wall Monolith TW-5. Ready for concrete lift 4.

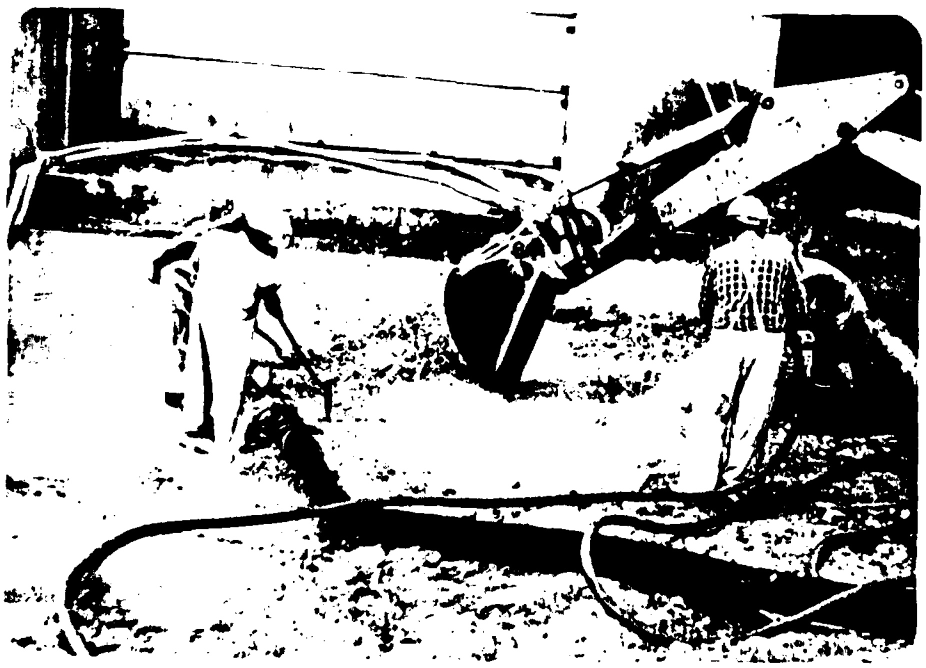


129. Harry S. Truman Dam, 9 November 1972. Neg. No. 965-17. Spillway-Powerhouse. Spillway training wall Monolith SW-5. Looking downstream. Ready for concrete lift 4.



130. Harry S. Truman Dam, 9 November 1972, Neg. No. 965-106. Spillway-Powerhouse. Spillway training wall Monolith SW-2. Looking downstream. Ready for concrete lift 4.

157. Harry S. Truman Dam, 13 April 1977, Neg. No. 77-122. Powerhouse draft tubes. Looking upstream.



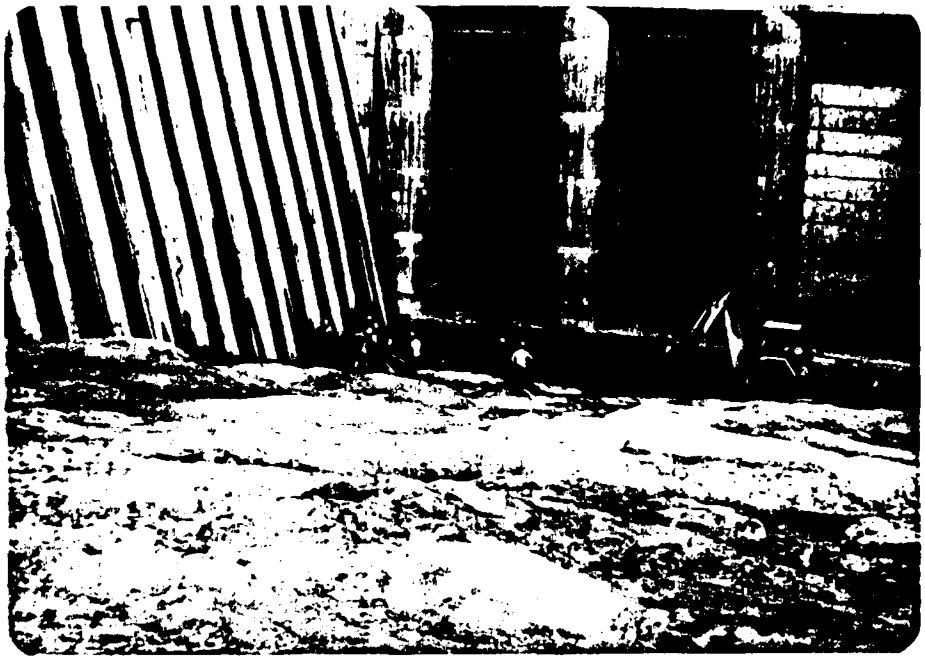
158. Harry S. Truman Dam, 13 April 1977, Neg. No. 77-123. Cleaning trairace floor. Downstream of draft tubes.

155. Harry S. Truman Dam, 11 April 1977, Neg. No. 77-120. Approach channel. Looking right and upstream.

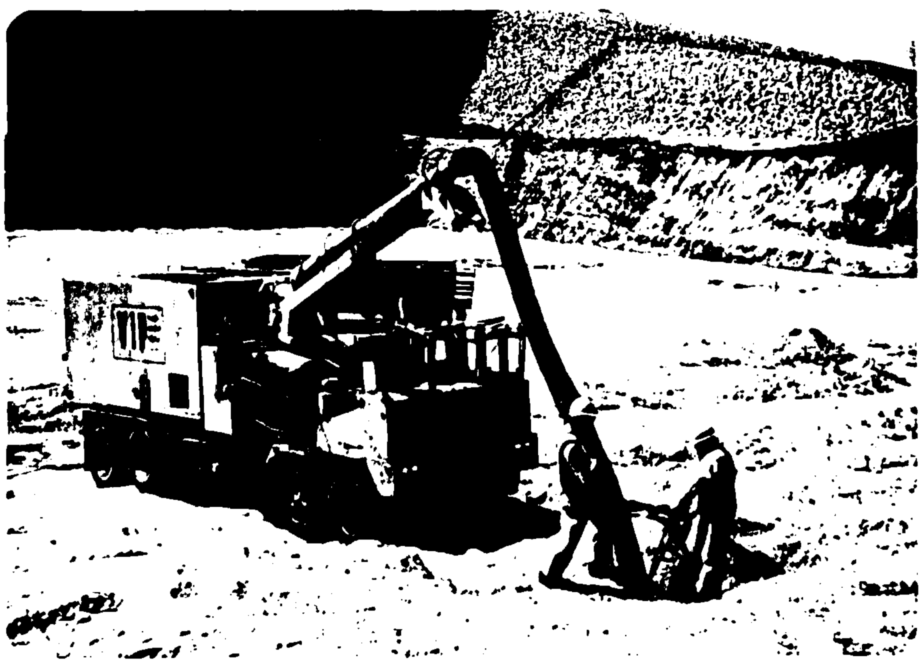
156. Harry S. Truman Dam, 13 April 1977, Neg. No. 77-121. Powerhouse draft tubes. Looking upstream.

153. Harry S. Truman Dam, 8 April 1977, Neg. No. 77-116. Approach channel, left side.

154. Harry S. Truman Dam, 11 April 1977, Neg. No. 77-117. Approach channel right side.



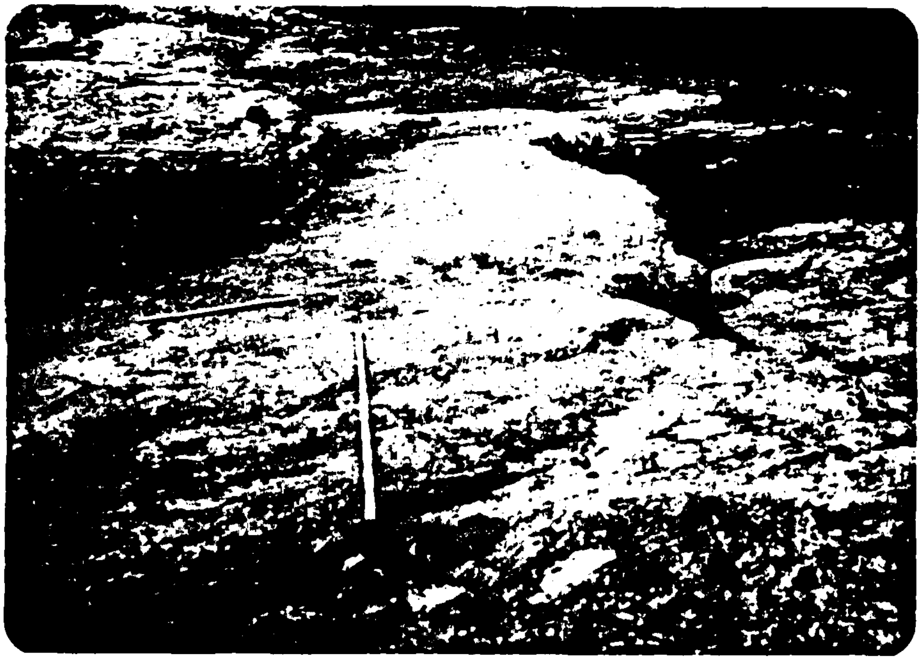
151. Harry S. Truman Dam, 7 April 1977, Neg. No. 77-114.
Tailrace floor, looking upstream.



152. Harry S. Truman Dam, 8 April 1977, Neg. No. 77-115.
Approach channel right side.

149. Harry S. Truman Dam, 1 April 1977, Neg. No. 77-111.
Downstream end of tailrace wall.

150. Harry S. Truman Dam, 1 April 1977, Neg. No. 77-113.
Downstream end of divider wall, looking left.

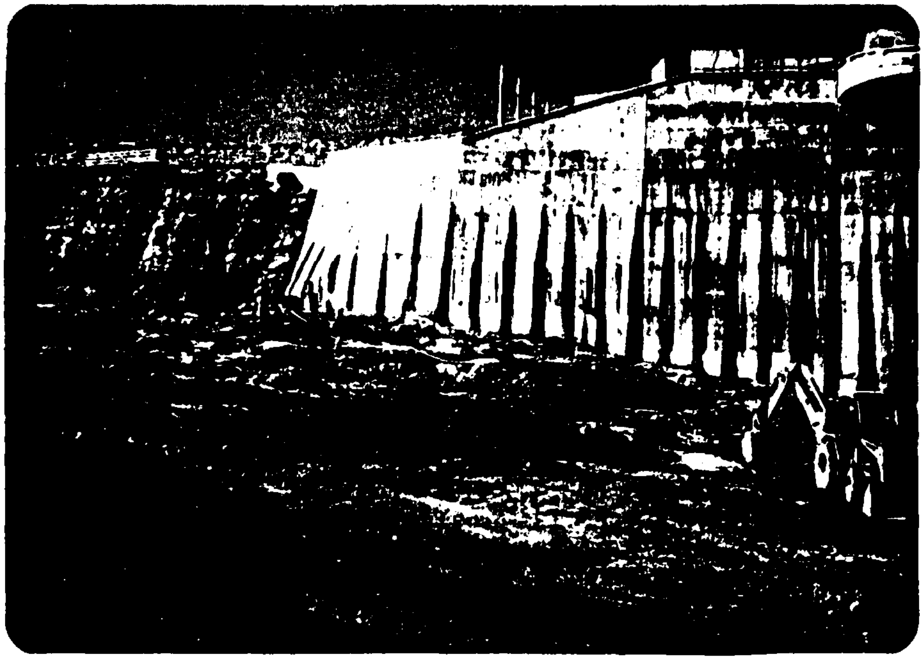


147. Harry S. Truman Dam, 1 March 1977, Neg. No. 77-104. Tailrace floor, looking downstream and to the right.

148. Harry S. Truman Dam, 1 April 1977, Neg. No. 77-110. Tailrace floor, looking downstream.

145. Harry S. Truman Dam, 25 March 1977, Neg. No. 77-59.
Tailrace floor, looking downstream.

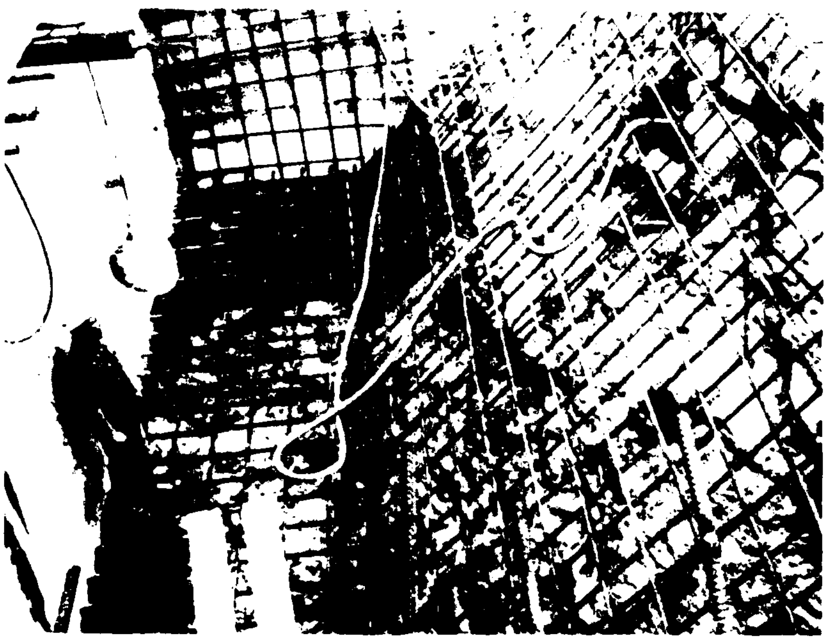
146. Harry S. Truman Dam, 29 March 1977, Neg. No. 77-68.
Tailrace floor, looking downstream.



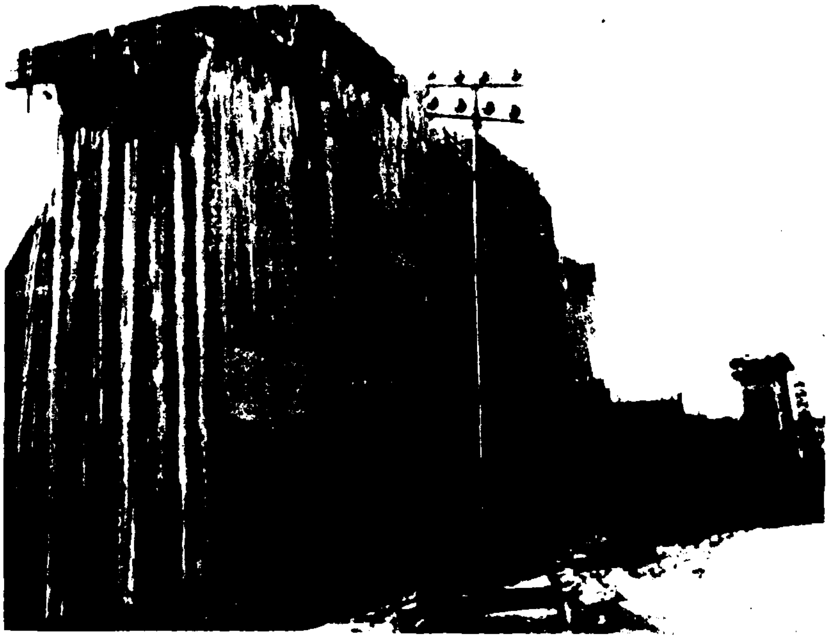
143. Harry S. Truman Dam, 25 March 1977, Neg. No. 77-53.
Downstream end of tailrace training wall

144. Harry S. Truman Dam, 25 March 1977, Neg. No. 77-56.
Cleaning surface of tailrace floor.

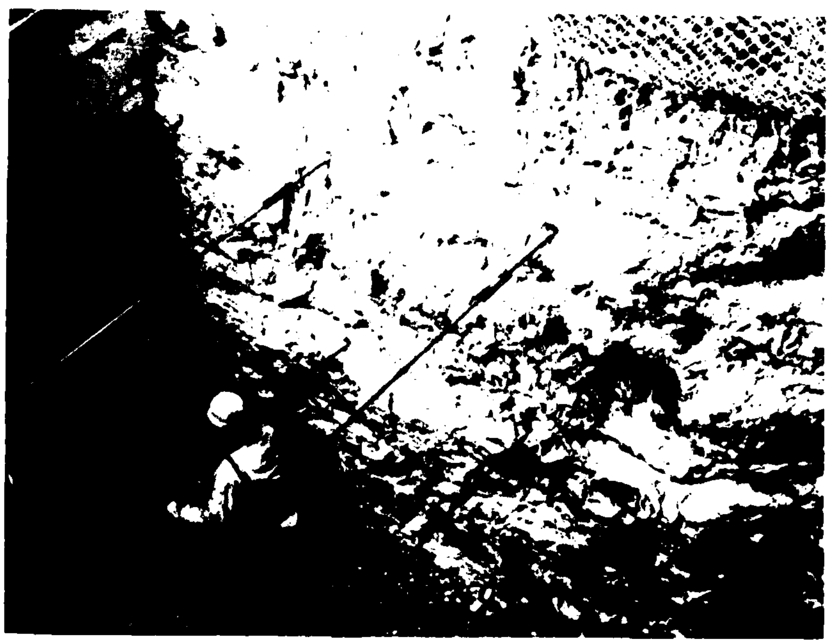
141. Harry S. Truman Dam, 21 March 1973, Neg. No. 965-65.
Spillway-Powerhouse. Tailrace training wall Monolith TW-5.
Ready for concrete lift 7.



142. Harry S. Truman Dam, 30 March 1972, Neg. No. 965-101
Spillway-Powerhouse. Downstream slope, 1V on 1H,
of Erection Bay, looking left.

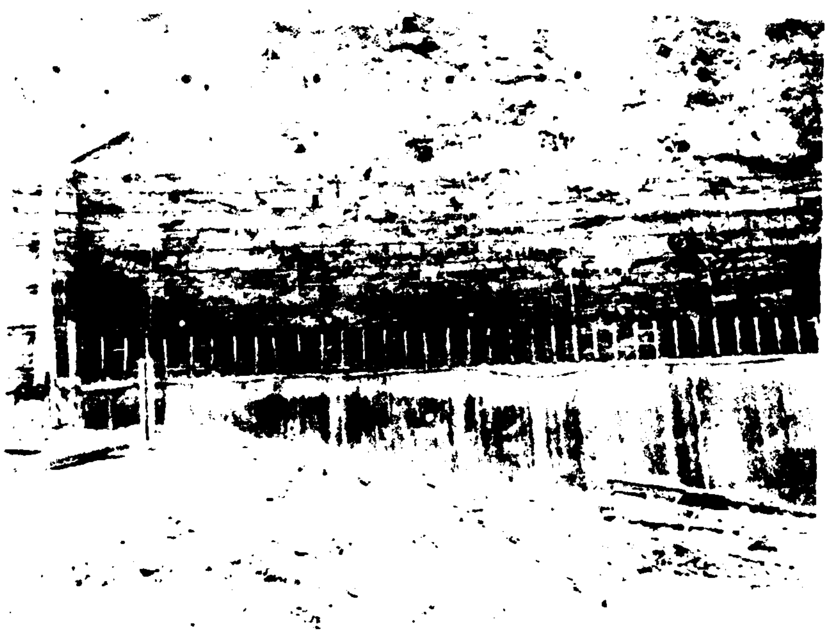


139. Harry S. Truman Dam, 6 December 1972, Neg. No. 965-100. Spillway-Powerhouse. Upstream side Monoliths 1 thru 8.

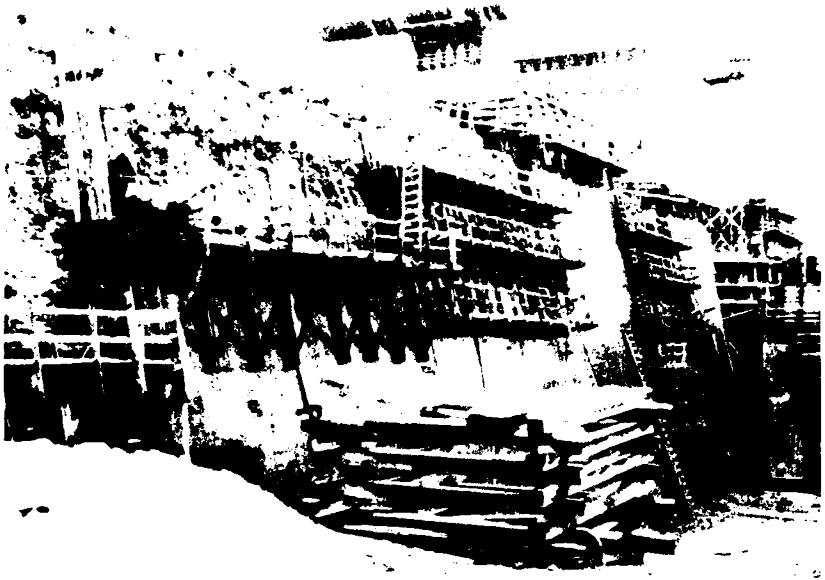


140. Harry S. Truman Dam, 1 February 1973, Neg. No. 965-102. Spillway-Powerhouse. Tailrace training wall Monolith TW-1. Ready for concrete lift 1.

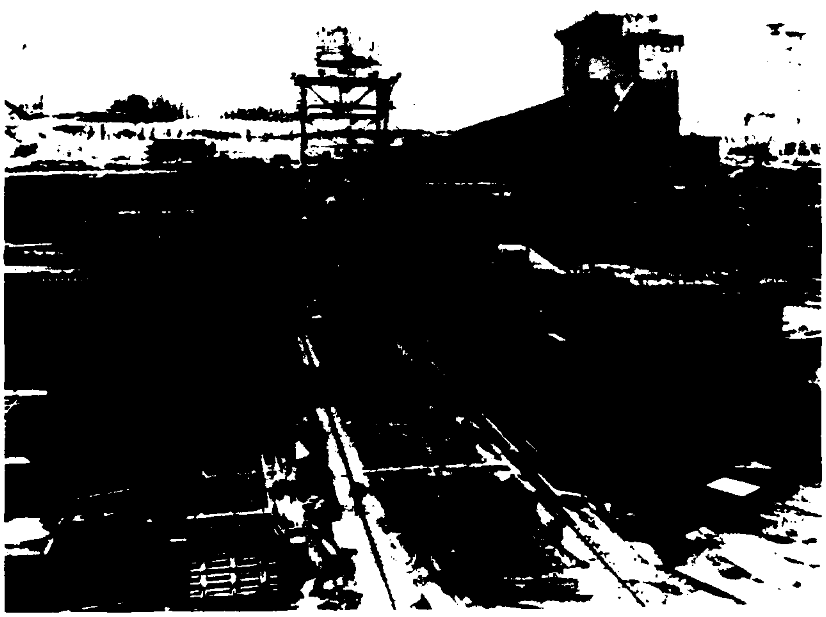
137. Harry S. Truman Dam, 20 November 1972, Neg. No. 965-97. Spillway-Powerhouse. View looking upstream Monoliths 12 thru 16.



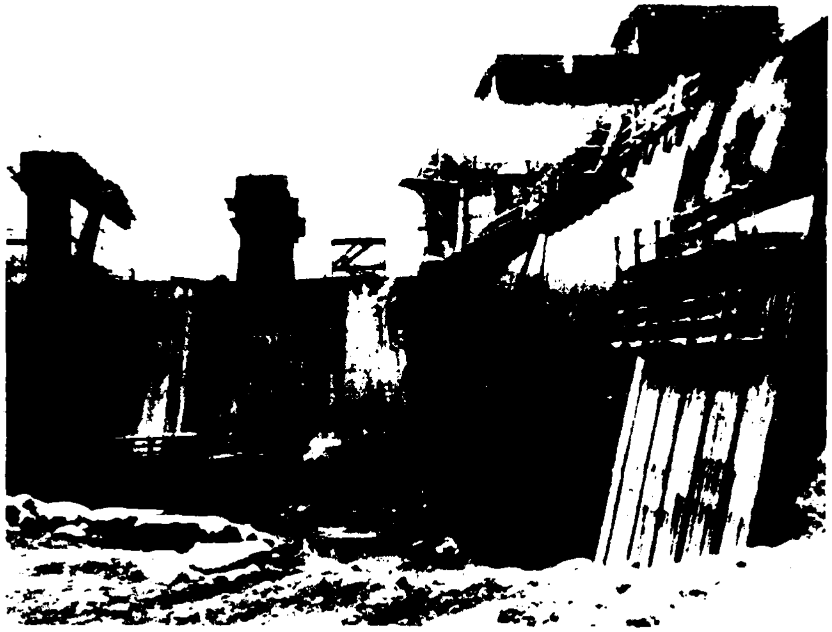
138. Harry S. Truman Dam, 27 November 1972, Neg. No. 965-25. Spillway-Powerhouse. Tailrace training wall Monolith TW-7. Ready for concrete lift 2.



135. Harry S. Truman Dam,
20 November 1972,
Neg. No. 965-119.
Spillway-Powerhouse.
Tailrace training
wall looking upstream.



136. Harry S. Truman Dam,
20 November 1972,
Neg. No. 965-1.
Spillway-Powerhouse.
Looking right from Sta
51+70, Control Line B.



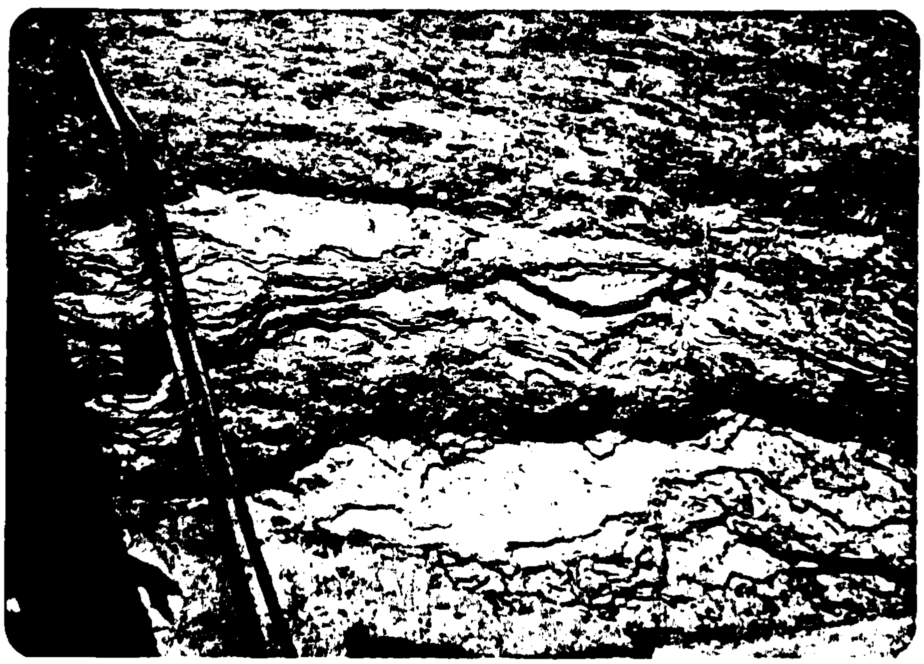
133. Harry S. Truman Dam, 20 November 1972, Neg. No. 965-109. Spillway-Powerhouse. Spillway training wall. Looking upstream.

134. Harry S. Truman Dam, 20 November 1972, Neg. No. 965-27. Spillway-Powerhouse. Spillway training wall Monolith SW-6.

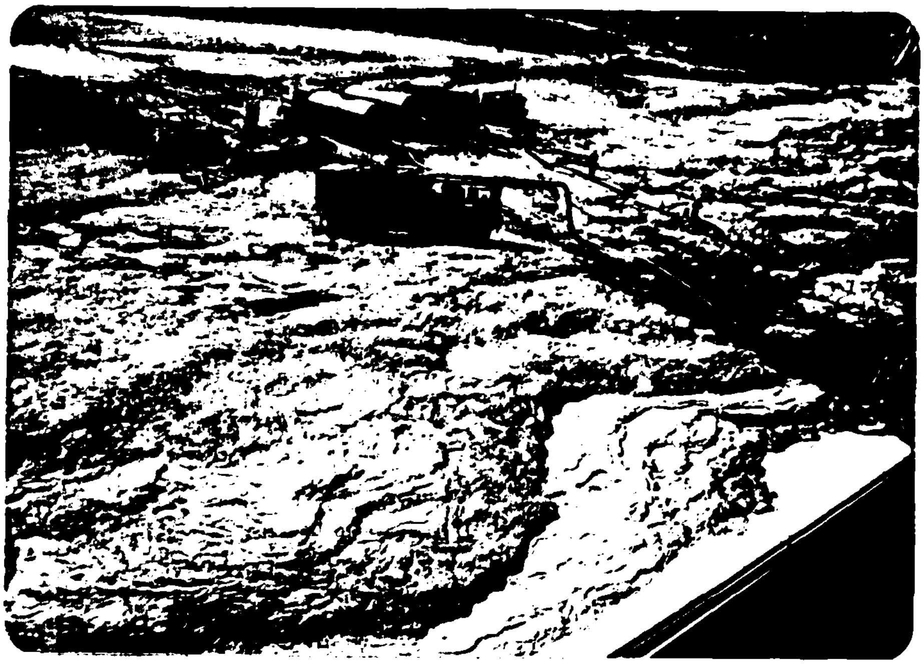
131. Harry S. Truman Dam, 13 November 1972, Neg. No. 965-108. Spillway-Powerhouse. Spillway training wall Monolith SW-4. Looking downstream. Cleanup for concrete lift 5.

132. Harry S. Truman Dam, 15 November 1972, Neg. No. 965-98. Spillway-Powerhouse. Tailrace training wall Monolith TW-5. Ready for concrete lift 5.

159. Harry S. Truman Dam, 14 April 1977, Neg. No. 77-124. Divider wall and 1V on 5H slope of tailrace floor.



160. Harry S. Truman Dam, 14 April 1977, Neg. No. 77-125. Tailrace floor. Looking downstream from top of divider wall.

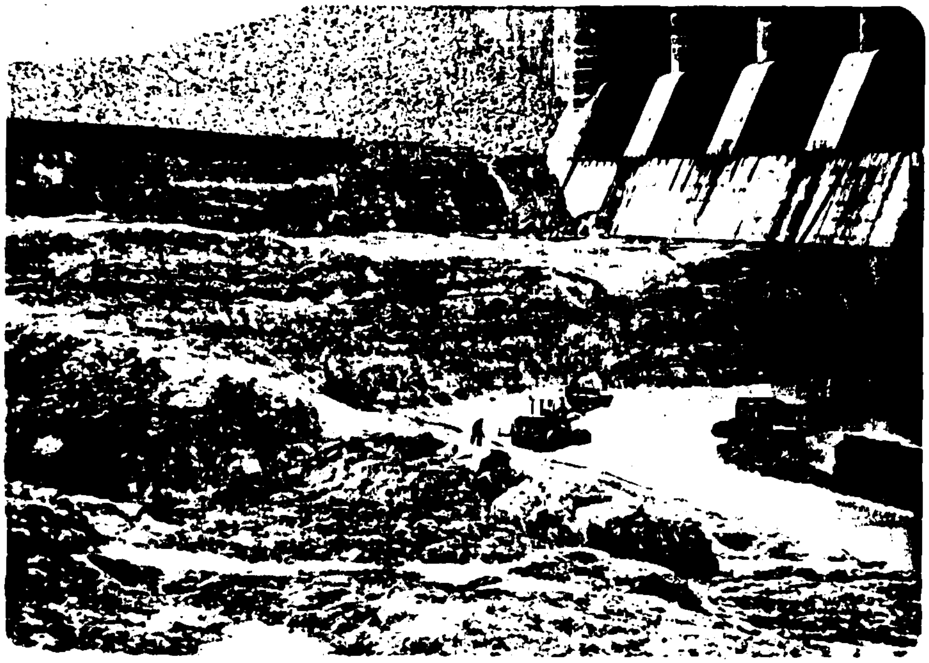


161. Harry S. Truman Dam, 14 April 1977, Neg. No. 77-127. Tailrace channel. Looking downstream from top of powerhouse.

162. Harry S. Truman Dam, 14 April 1977, Neg. No. 77-130. Tailrace channel. Looking downstream from top of powerhouse.

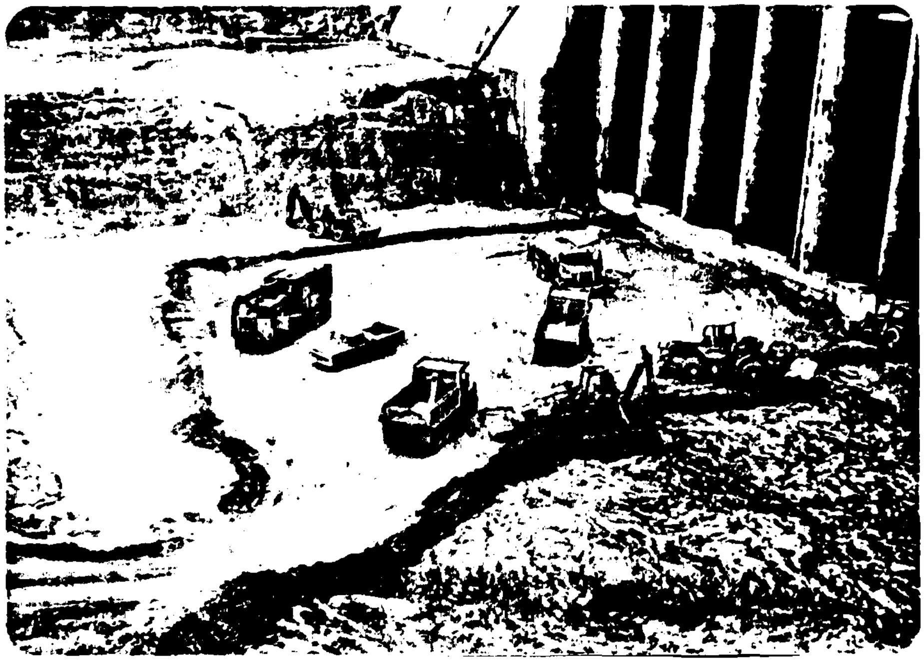
163. Harry S. Truman Dam, 22 April 1977, Neg. No. 77-148. Spillway and tailrace floor. Looking downstream from top of powerhouse.

164. Harry S. Truman Dam, 26 April 1977, Neg. No. 77-151. Spillway approach channel. Looking upstream and to the left. 1V on 5H.



165. Harry S. Truman Dam, 3 May 1977, Neg. No. 77-196. Upstream face of spillway and left well of approach channel.

166. Harry S. Truman Dam, 3 May 1977, Neg. No. 77-199. Upstream face of spillway and left wall of approach channel.



167. Harry S. Truman Dam, 6 May 1977, Neg. No. 77-216. Powerhouse intake.

168. Harry S. Truman Dam, 6 May 1977, Neg. No. 77-219. Powerhouse approach channel.

169. Harry S. Truman Dam, 25 May 1977, Neg. No. 553-R1-22. Aerial View Osage River looking upstream before diversion and closure.

170. Harry S. Truman Dam, 25 May 1977, Neg. No. 553-R2-1. View of embankment from left abutment.

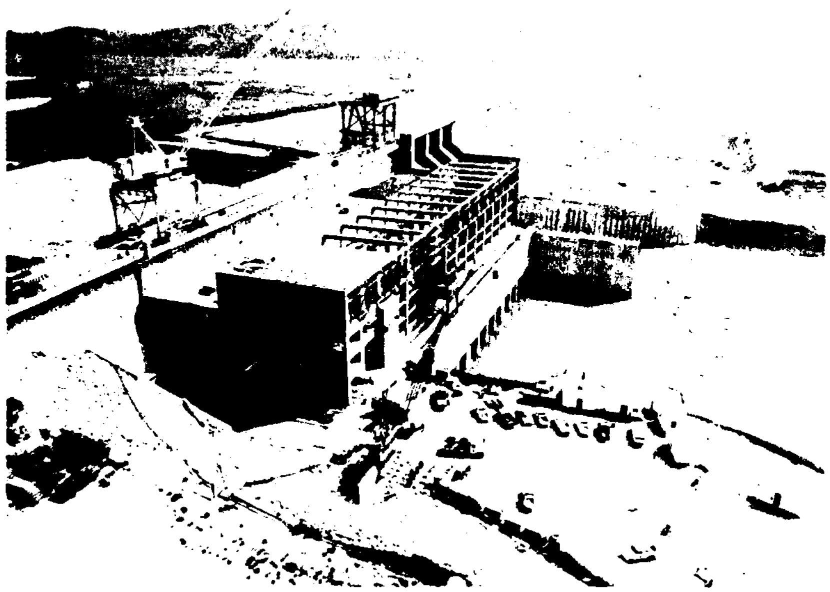
169. Harry S. Truman Dam, 25 May 1977, Neg. No. 553-R1-22. Aerial View Osage River looking upstream before diversion and closure.

170. Harry S. Truman Dam, 25 May 1977, Neg. No. 553-R2-1. View of embankment from left abutment.

171. Harry S. Truman Dam, 25 May 1977, Neg. No. 553-R2-5. View of embankment from near toe of left abutment.

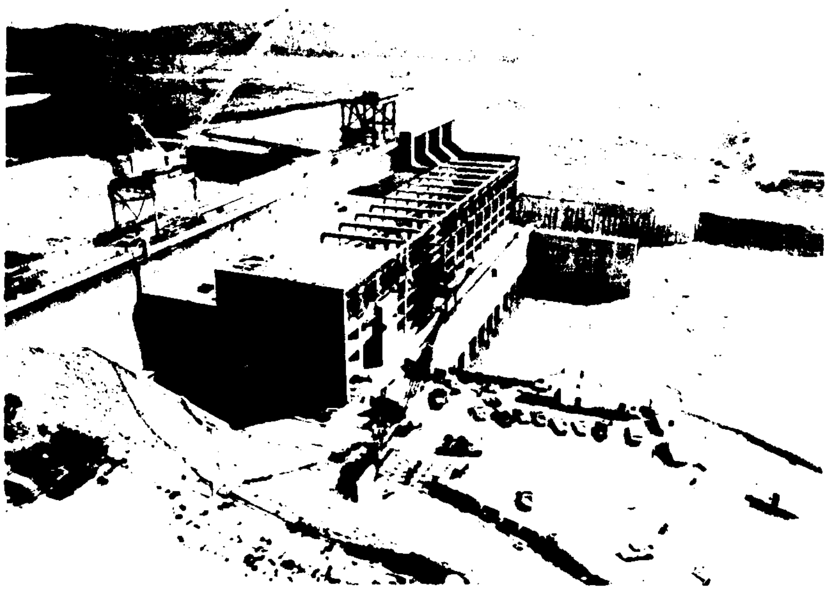
172. Harry S. Truman Dam, 25 May 1977, Neg. No. 553-R2-8. Outlet channel. Contractor's bridge under construction.

173. Harry S. Truman Dam, 25 May 1977, Neg. No. 553-R2-10.
Outlet channel.



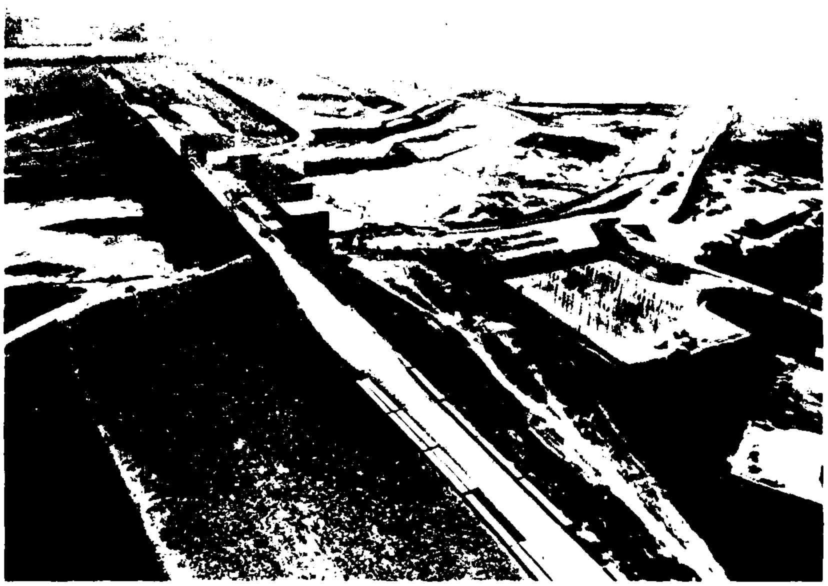
174. Harry S. Truman Dam, 25 May 1977, Neg. No. 553-R1-11.
Spillway-Powerhouse. View from right abutment.

173. Harry S. Truman Dam, 25 May 1977, Neg. No. 553-R2-10. Outlet channel.

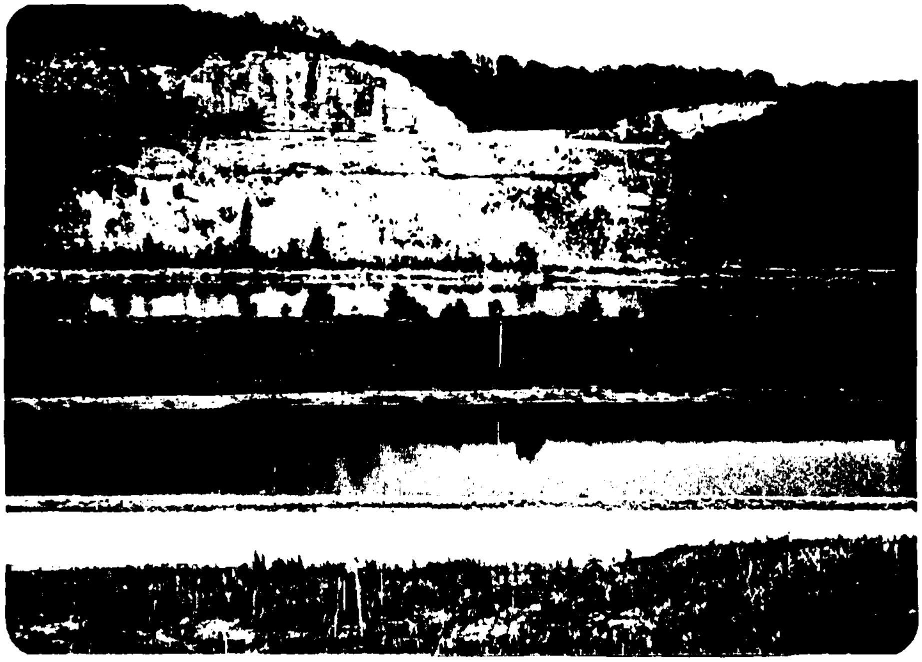


174. Harry S. Truman Dam, 25 May 1977, Neg. No. 553-R1-11. Spillway-Powerhouse. View from right abutment.

175. Harry S. Truman Dam, 25 May 1977, Neg. No. 553-R2-14.
Left abutment from dam axis Sta. 62+00.



176. Harry S. Truman Dam, 25 May 1977, Neg. No. 553-R2-15.
Aerial view of embankment and spillway-powerhouse.



177. Harry S. Truman Dam, 20 May 1977, Neg. No. 77-235. Outlet channel downstream of channel plug. Osage River and left abutment in background.

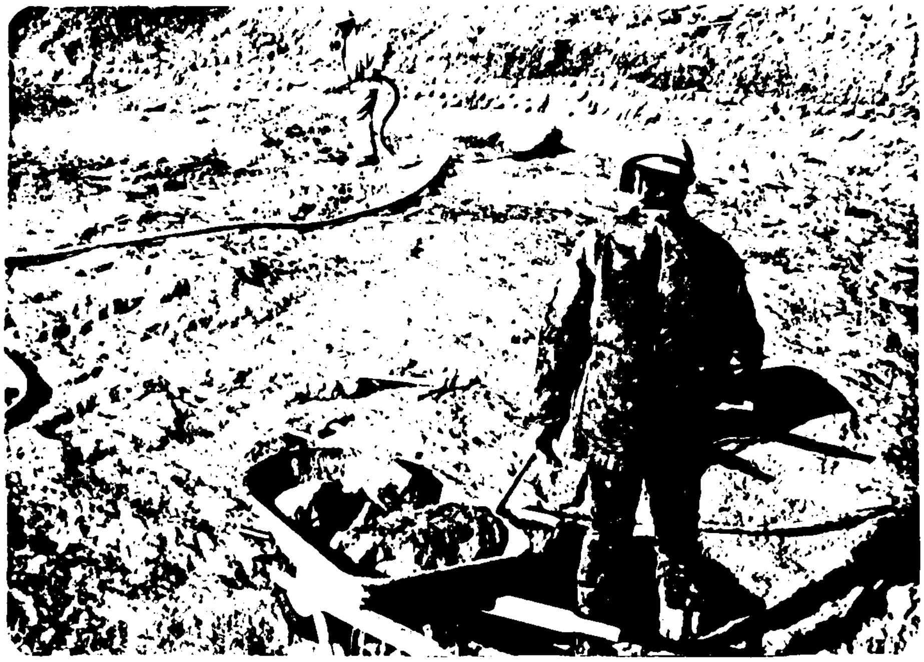
178. Harry S. Truman Dam, 21 June 1977, Neg. No. 77-322. Approach channel and upstream face of spillway.

179. Harry S. Truman Dam, 22 June 1977, Neg. No. 77-326. Left wall spillway outlet channel just downstream of spillway training wall.

180. Harry S. Truman Dam, 22 June 1977, Neg. No. 77-327. Spillway outlet channel. Looking downstream.

181. Harry S. Truman Dam, 22 June 1977, Neg. No. 77-328. Spillway outlet channel. Looking downstream.

182. Harry S. Truman Dam, 22 June 1977, Neg. No. 77-331. Approach channel. Looking to right. Note concrete dental work on floor and left side rock slope 1V on .75H.



208. Harry S. Truman Dam, 25 November 1977, Neg. No. 77-719. Closure area cutoff trench, Sta. 63+60. Cleaning foundation surface.

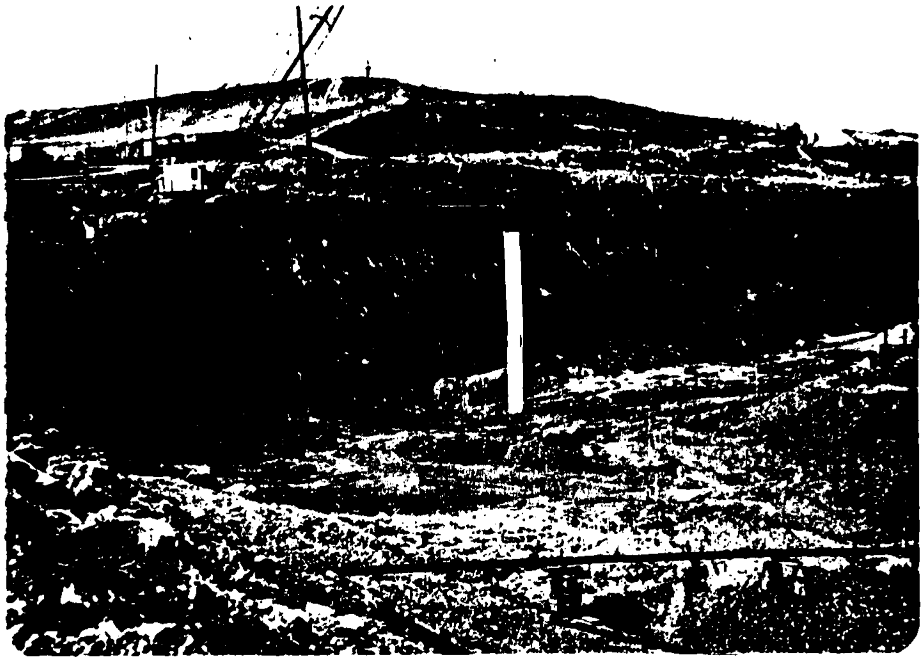
209. Harry S. Truman Dam, 25 November 1977, Neg. No. 77-720. Closure area cutoff trench, Sta. 63+45. Placing impervious clay.

206. Harry S. Truman Dam, 11 November 1977, Neg. No. 77-702. Closure area cutoff trench, Sta. 63+10. Cleaning foundation surface.

207. Harry S. Truman Dam, 25 November 1977, Neg. No. 77-718. Closure area cutoff trench, Sta. 63+50. Placing impervious clay in open joints.

204. Harry S. Truman Dam, 11 November 1977, Neg. No. 77-669. Closure area Sta. 63+90. Cleaning foundation surface. Looking downstation.

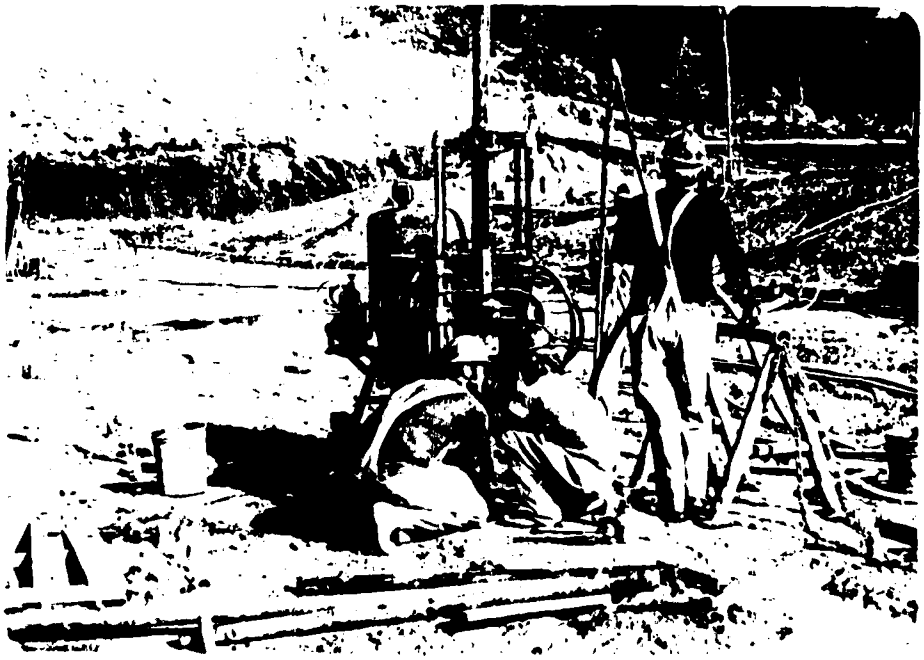
205. Harry S. Truman Dam, 11 November 1977, Neg. No. 77-701. Closure area cutoff trench, Sta. 63+40. Placing impervious clay in open joints.



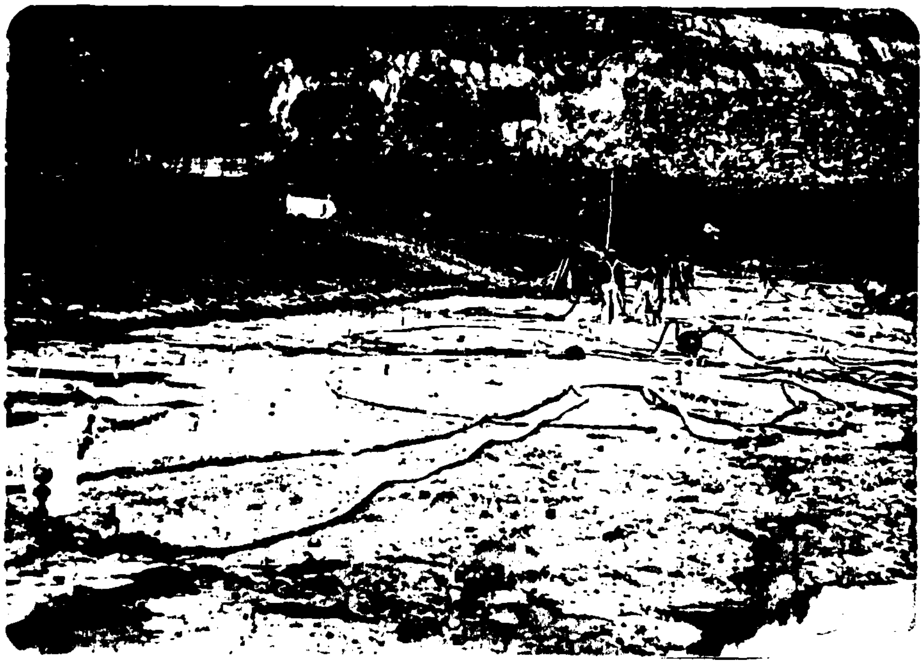
202. Harry S. Truman Dam, 29 October 1977, Neg. No. 77-632. Closure area, 36 inch sump at Sta. 66+50, range 500D. Looking upsteam and to the right

203. Harry S. Truman Dam, 11 November 1977, Neg. No. 77-668. Closure area Sta. 62+90 cutoff trench. Cleaning foundation surface.

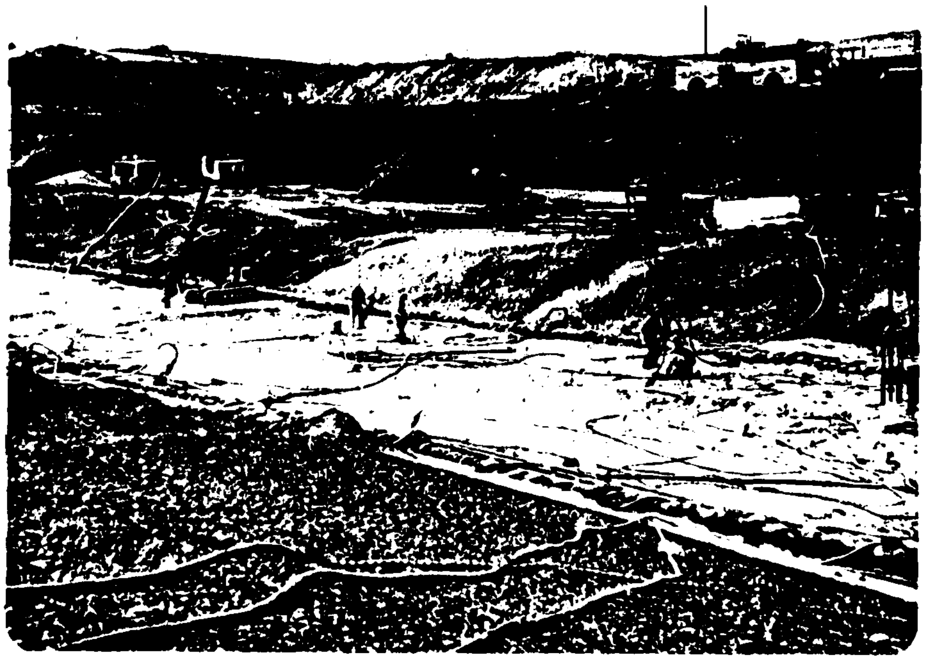
200. Harry S. Truman Dam, 30 October 1977, Neg. No. 77-627. Closure area, pressure testing before grouting.



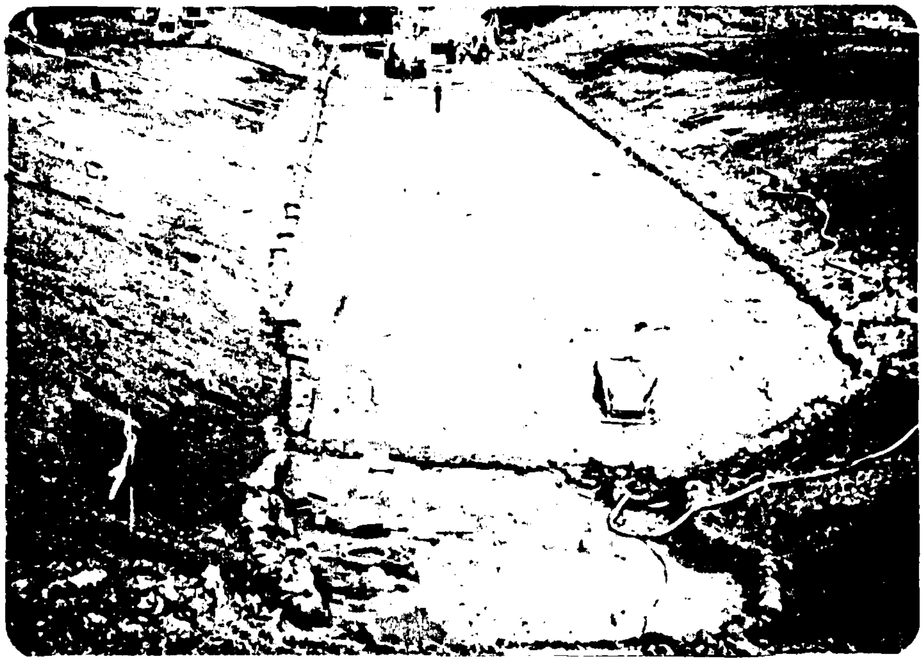
201. Harry S. Truman Dam, 26 October 1977, Neg. No. 77-629. Closure area, cutoff trench Sta. 63+90. Drilling exploratory core hole C-1, 4 feet upstream of dam axis.



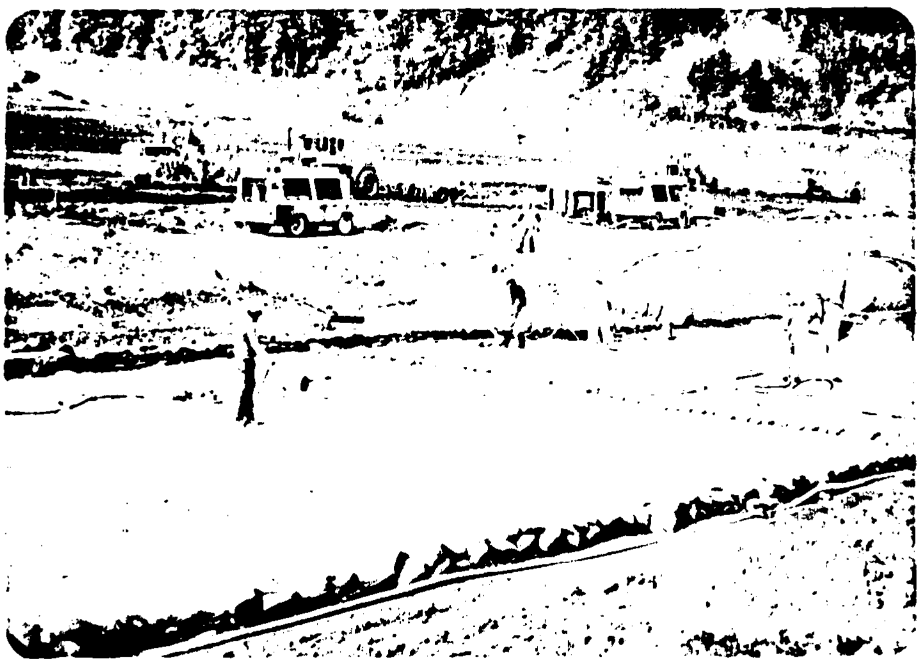
198. Harry S. Truman Dam, 6 October 1977, Neg. No. 77-584. Closure area, cutoff trench grouting in Line C on Dam axis, Sta. 67+50.



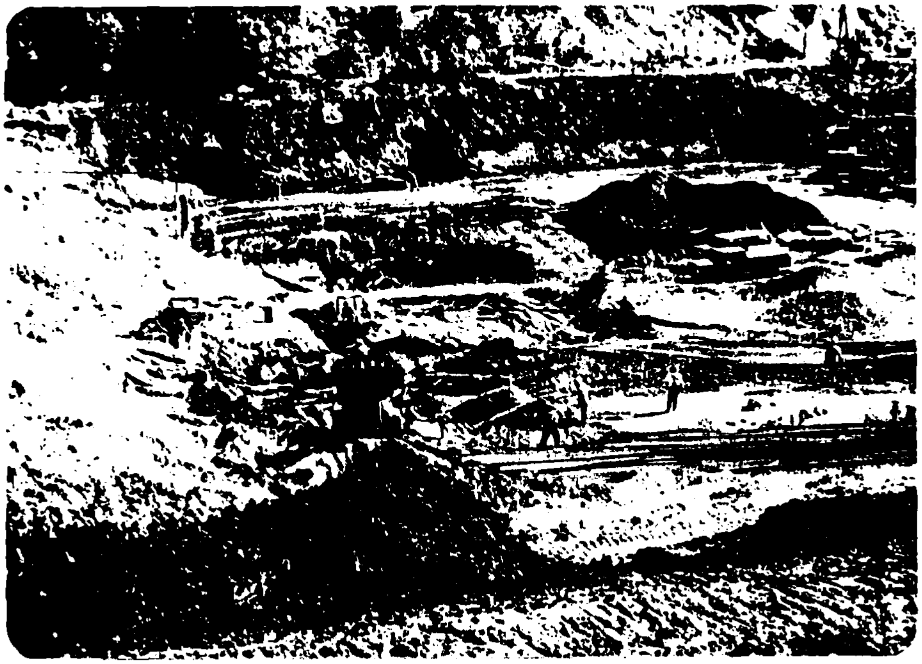
199. Harry S. Truman Dam, 30 October 1977, Neg. No. 77-607. Closure area, cutoff trench. Grouting on Line A, 10 feet upstream of dam axis Sta. 68+65. Looking upstream and to the right.



196. Harry S. Truman Dam, 6 October 1977, Neg. No. 77-566. Closure area, cutoff trench Sta. 62+70. Looking upstation.



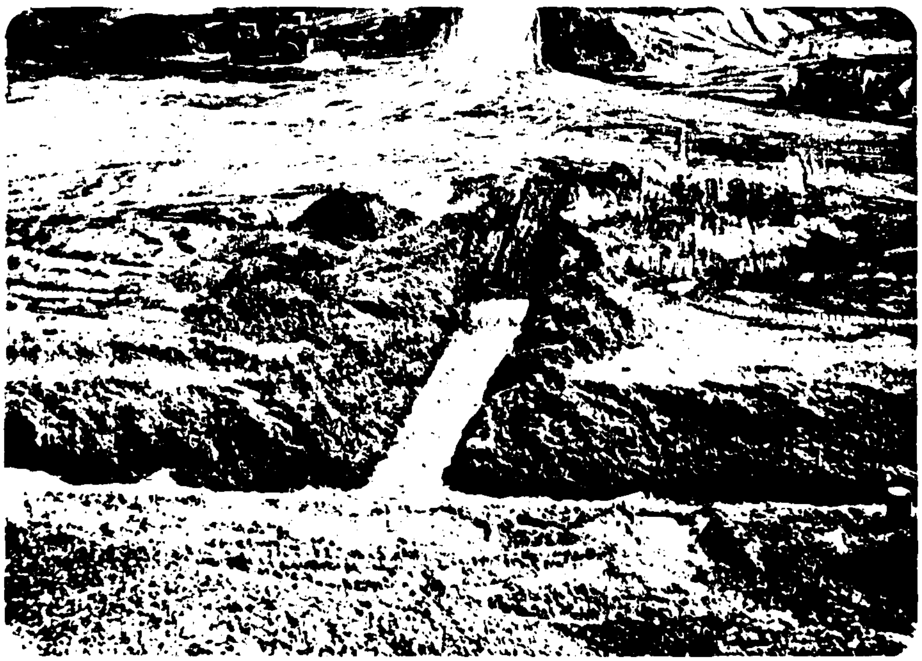
197. Harry S. Truman Dam, 6 October 1977, Neg. No. 77-567. Closure area, cutoff trench Sta 64+40. Looking upstream and toward left abutment.



194. Harry S. Truman Dam, 22 September 1977, Neg. No. 77-544. Closure area. Looking toward left abutment.

195. Harry S. Truman Dam, 22 September 1977, Neg. No. 77-534. Dewatering trench in basal gravel before installation of collector pipe.

192. Harry S. Truman Dam, 22 September 1977, Neg. No. 77-535.
Dewatering trench in basal gravel, closure area.



193. Harry S. Truman Dam, 22 September 1977, Neg. No. 77-537.
Dewatering trench in basal gravel, closure area.

190. Harry S. Truman Dam, 16 September 1977, Neg. No. 965-19. Upstream rockfill coffer dam, closure area.

191. Harry S. Truman Dam, 1 October 1977, Neg. No. 965-20. Closure area. Basal gravel toe of left abutment.

188. Harry S. Truman Dam, 3 September 1977, Neg. No. 965-23. Muck excavation, closure area. Looking upstream from left abutment.

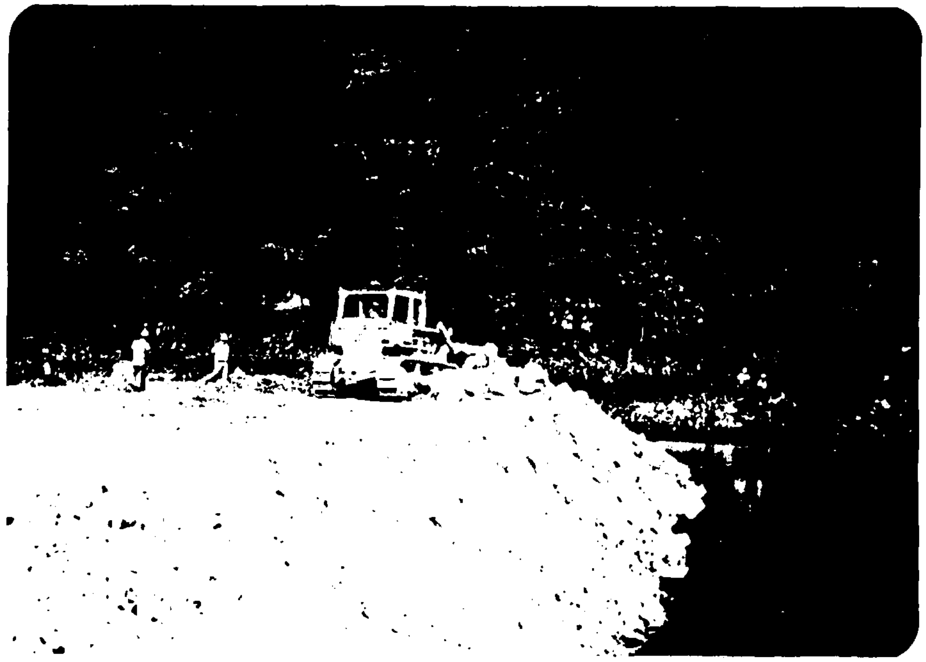
189. Harry S. Truman Dam, 6 September 1977, Neg. No. 965-110. Cleanup, closure area. Looking downstream.

187. Harry S. Truman Dam, 27 August 1977, Neg. No. 965-127. View of closure area from left abutment.

185. Harry S. Truman Dam, 29 July 1977, Neg. No. 77-429. Closure area. Excavating muck. Looking upstream.

186. Harry S. Truman Dam, 2 September 1977, Neg. No. 77-502. Muck excavation, closure area. Looking upstream at Sta. 69+50.

12



183. Harry S. Truman Dam, 21 July 1977, Neg. No. 77-390. Constructing upstream rock fill coffer dam for closure. Looking at left abutment.

184. Harry S. Truman Dam, 21 July 1977, Neg. No. 77-392. Constructing upstream rock fill coffer dam. Looking upstream along toe of left abutment.

210. Harry S. Truman Dam, 21 April 1978, Neg. No. 965-90. Closure area. Looking toward left abutment from Sta. 65+30.

211. Harry S. Truman Dam, 21 April 1978, Neg. No. 965-82. Closure area Sta. 65+30.

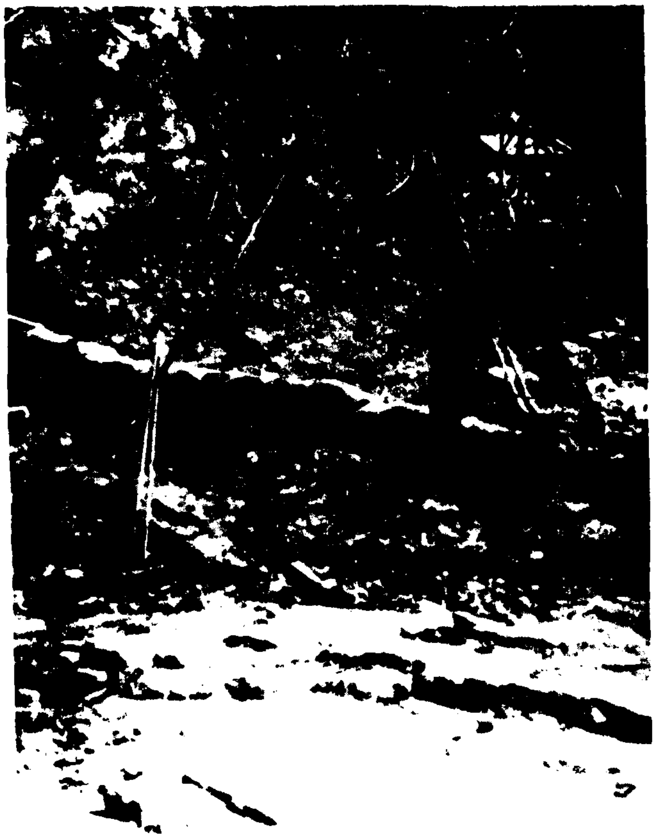
212. Harry S. Truman Dam, 25 April 1978, Neg. No. 965-80. Closure area cutoff trench Sta. 65+00. Looking downstation.

213. Harry S. Truman Dam, 25 April 1978, Neg. No. 965-46. Closure area cutoff trench Sta. 67+00. Looking upstream.

214. Harry S. Truman Dam, 1 May 1978, Neg. No. 965-126. View of closure area from left abutment.

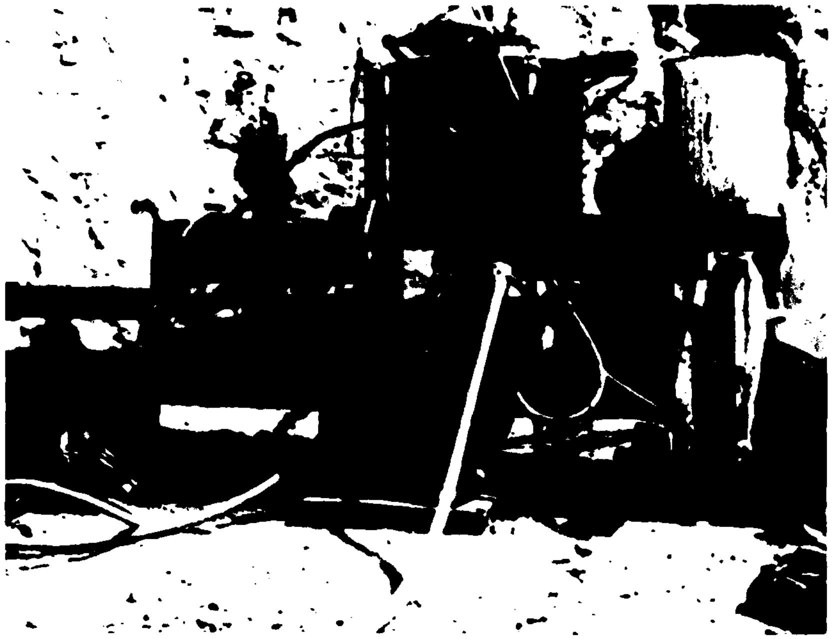
215. Harry S. Truman Dam, 30 May 1978, Neg. No. 965-112. Closure area cutoff trench Sta. 69+50. Hand tamping impervious clay on breccia bedrock surface.

216. Harry S. Truman Dam, 30 May 1978, Neg. No. 965-13. Closure area cutoff trench Sta. 69+75.



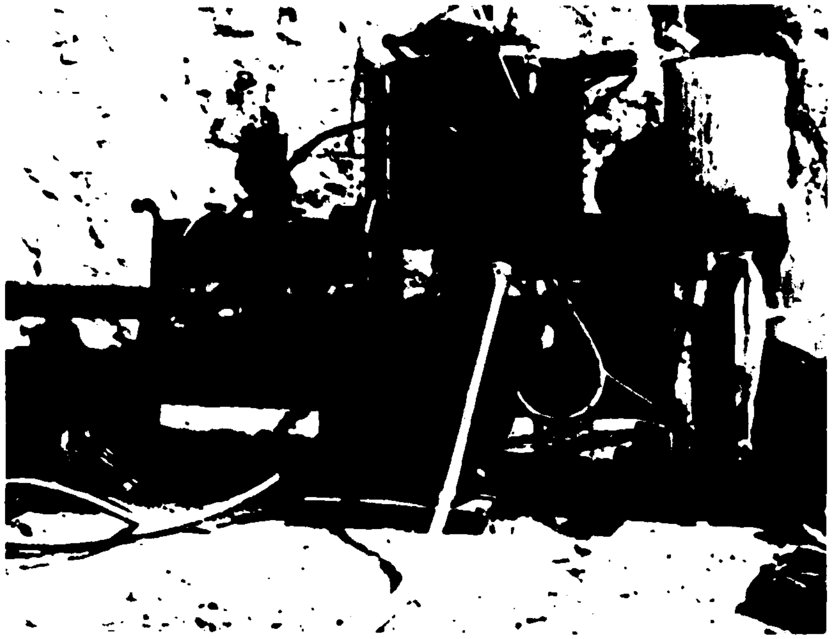
217. Harry S. Truman Dam, 30 May 1978, Neg. No. 965-121. Closure area cutoff trench at toe of left abutment Sta. 70+00 to Sta. 70+50. Pressure testing on grout Line C, on dam axis.

218. Harry S. Truman Dam, 14 June 1978, Neg. No. 965-114. Grouting on left abutment, Sta. 70+00.



219. Harry S. Truman Dam, 15 June 1978, Neg. No. 965-51.
Grout plant on left abutment.

220. Harry S. Truman Dam, 15 June 1978, Neg. No. 965-6.
Placing impervious clay against left abutment.



219. Harry S. Truman Dam, 15 June 1978, Neg. No. 965-51.
Grout plant on left abutment.

220. Harry S. Truman Dam, 15 June 1978, Neg. No. 965-6.
Placing impervious clay against left abutment.

221. Harry S. Truman Dam, 26 June 1978, Neg. No. 965-41.
Placing impervious clay against left abutment.

222. Harry S. Truman Dam, 3 July 1978, Neg. No. 965-10.
Looking down left abutment at Sta. 72+80, Range 100D.

223. Harry S. Truman Dam, 3 July 1978, Neg. No. 965-52. Toe of left abutment, Sta. 70+50, Range 0+50D.

224. Harry S. Truman Dam, 3 July 1978, Neg. No. 965-11. Toe of left abutment Sta. 70+50, Range 1+00D.

225. Harry S. Truman Dam, 3 July 1978, Neg. No. 965-7. Impervious clay against toe of left abutment Sta. 70+50, Range 1+75D.

226. Harry S. Truman Dam, 3 July 1978, Neg. No. 965-49. Toe of left abutment Sta. 70+50. Drain at Range 1+75D. Bottom el. 640±.

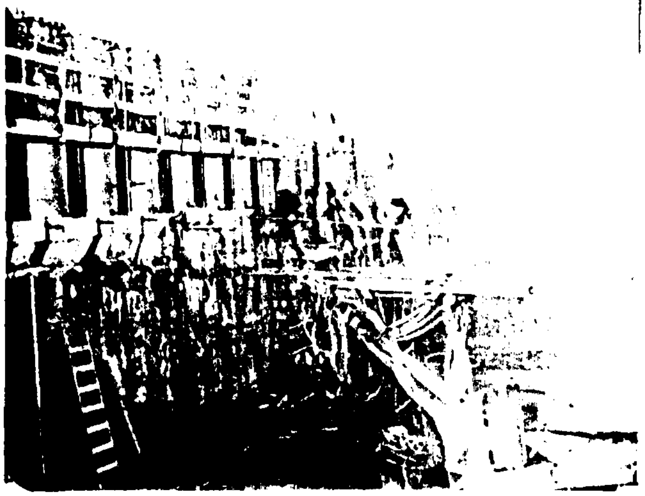
227. Harry S. Truman Dam, 20 October 1978, Neg. No. 965-128, View of closure area from left abutment U/S El. 662.6, D/S El. 657.2.

228. Harry S. Truman Dam, 12 June 1975, Neg. No. 965-55.
Sterett Creek Dike, cutoff trench. Right abutment
Sta. 10+00. Looking south.

229. Harry S. Truman Dam, 12 June 1975, Neg. No. 965-75.
Sterett Creek Dike, cutoff trench. Contact of units
15/16. Sta. 11+20.

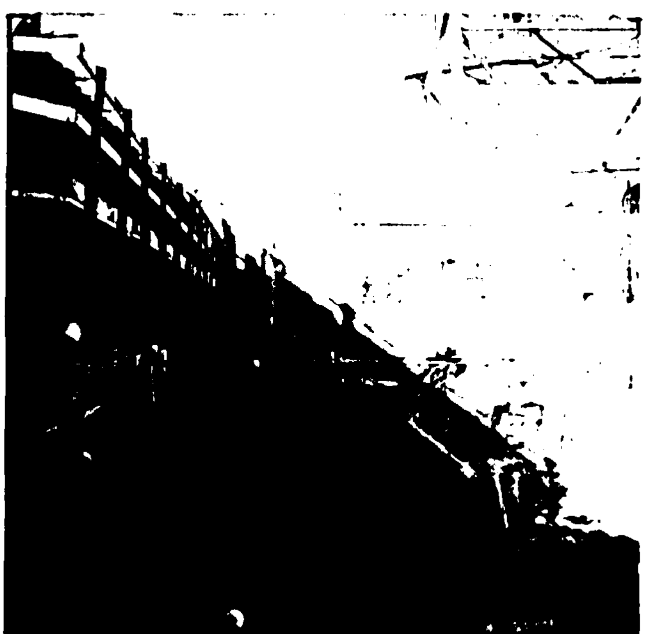
230. Harry S. Truman Dam, 12 June 1975, Neg. No. 965-74. Sterett Creek Dike, cutoff trench. Unit 16. Looking upstream, Sta. 11+30.

231. Harry S. Truman Dam, 10 July 1975, Neg. No. 965-15. Sterett Creek Dike, inspection trench. Looking upstream at Sta. 75+00.



232. Harry S. Truman Dam, 18 February 1972, Neg. No. 72-44. Installing Perfo. sleeve rock bolts on upstream face of powerhouse wall.

233. Harry S. Truman Dam, 18 February 1972, Neg. No. 72-45. Mixing mortar for Perfo. sleeve rock bolts.



234. Harry S. Truman Dam, 18 February 1972, Neg. No. 72-48. Installing Perfo. sleeve rock bolts on upstream face of powerhouse wall.

AD-A154 209 MULTIPLE-PURPOSE PROJECT OSAGE RIVER BASIN OSAGE RIVER 3/3
MISSOURI HARRY S T..(U) CORPS OF ENGINEERS KANSAS CITY
MO KANSAS CITY DISTRICT R F GRIFFITH ET AL. 1984

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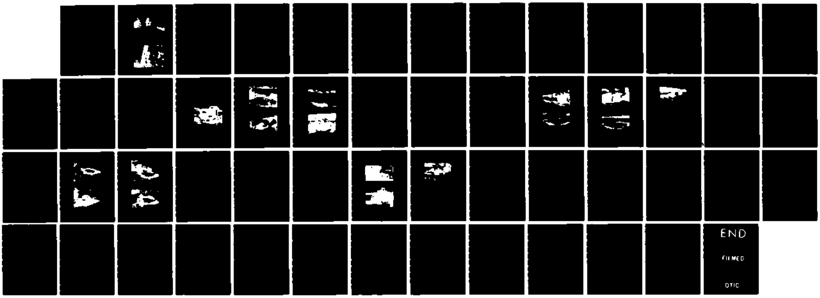

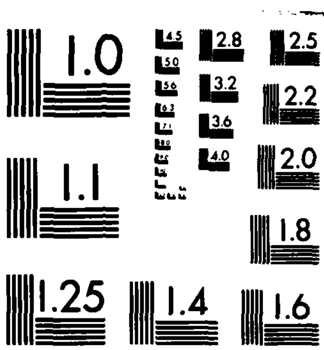

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

235. Harry S. Truman Dam, 16 August 1973, Neg. No. 73-331.
Percolation test on embankment.

236. Harry S. Truman Dam, 2 July 1973, Neg. No. 73-272.
Pull out test on rock bolts.

SUPPLEMENTS

SUPPLEMENT A

ROCK GRADATION TEST PROGRAM

1-01. Introduction: The economic feasibility of constructing a civil works project in a given region is based on the availability of the building materials. As to an earth and rock fill dam, the available materials need to possess specific inherent properties which will contribute to the overall integrity of the dam. At Harry Truman Dam, engineering studies have defined the proper rock sizes as one of the required properties and instituted a test program within the specifications to assure that the rock met the acceptable size requirements. Table A-1 is a summary of the test program.

Table A-1 Gradation Tests Required

	Required Tests	
	Truck Load	In Place
Type "A" riprap	1	1
Type "B" riprap	1	1
Spalls	1	
Bedding		2
Channel Scalped Rock		2
Channel Choker Course		2
Upstream Rock Slope Protection		6
Choker Course Zone		6
Scalped Rock Zone		6
Rock Zone 1		4
Rock Zone 2		4

1-02. Rock Products: The following rock products were produced from required excavation.

a. Type "A" Riprap: Protective rock placed on the left bank of the discharge channel from the downstream plug to the bare rock cut toward the Powerhouse.

b. Type "B" Riprap: Protective rock placed on the right bank of the discharge channel from the downstream plug to the bare rock cut toward the Powerhouse.

c. Spalls: A 1" bed of rock underlaying the spalls.

d. Bedding: A 6" bed of rock underlaying the spalls.

e. Channel Scalped Rock: An 18" protective layer of rock along both sides of the discharge channel from the downstream plug to the river.

f. Channel Choke Course: A 12" bed of rock underlaying the channel scalped rock.

g. Upstream Rock Slope Protection: A 2' protective rock layer on the upstream side of the rock dam embankment from natural ground (toe trench) to El. 717 to the right of the spillway - powerhouse structure and from natural ground to El. 703.5 to the left of the spillway - powerhouse structure.

h. Choker Course Zone: The 2' layer of rock placed atop the horizontal drain and in respective choker course zones used in the tie-ins with the concrete non overflow structures. Choker course rock was also used behind the tailrace training wall above top of rock, around the cable tunnel, downstream of the gravity wall, and in the parkng area on the spillway side of the concrete structure.

i. Scalped Rock Zone: A 1.5' layer forming a part of the horizontal drain downstream of the pervious rock zone and as a part of the tie-in with the concrete non overflow bulkheads.

j. Rock Zone 1: A rock mass section of the dam embankment upstream of the impervious core consisting of general run (quarry run) rock from the spillway powerhouse excavation.

k. Rock Zone 2: A 9' layer of the horizontal drain and a mass section from El. 740 to the crest on the upstream side of the embankment atop rock Zone 1. Rock Zone 2 also lies adjacent to rock Zone 3 on the non overflow bulkhead tie-ins on the upstream sides. Rock Zone 2 is exposed on the downstream side of the tie-ins.

l. Rock Zone 3: A 4' layer of the Burlington rock slope protection facing the approach channel.

m. Transition: A 10' wide zone of rock between the horizontal drain and the pervious zone.

1-03. Rock Zones 1, 2, and 3 Placement Procedures:

All hauling and placing of materials within Rock Zones 1, 2, and 3 were performed with rubber-tired equipment. The fine and less durable rock was placed alongside the berm and impervious zones. The larger and more durable sizes were distributed away from the impervious zones by manipulations of the rock rake.

In the outer 50 feet of rock Zone 1, above El. 701.5, the rock was coarser by selective loading of required excavation. Each load was dumped in line parallel to the dam axis in the zone. The rock rake spread the stockpiles outward toward the upstream edge normal to the dam axis. A gradational transition existed as a consequence of the operation. Larger rock (ones 10 to 12 inches) rests within the outer 20 feet of the zone and the finer rock within the interior.

1-04. Rock Product Gradations:

Table A-1 is a list of the gradation tests required under Stage III contract. In many cases more than the required number were performed and documented as part of quality assurance during production of a particular type of rock.

Tables A-2, 3, 4, 5, 6, 7, 8, 9 show the results of the required gradation tests.

Table A-2 Type "A" and Type "B" Riprap Gradation Tests

Weight	"A" Spec. Limit	"B" Spec. Limit	1	2	3	4
4,000#	100		100	100		
3,500#	85 - 95		88.4	94		
2,500#		100			100	100
2,000#		85 - 95			87.4	88
1,000#	35 - 50		38.7	49		
500#		30 - 50			47.5	49
100#	0 - 15		11.1	10		
50#		0 - 15			14.4	9
-100#						
-50#						

1. Type "A" riprap sampled from temporary stockpiles tested on 15 May 1974.

2. Type "A" riprap, in place sample Sta. 63+00, 75' Left "B" line.

3. Type "B" riprap, truck load sample.

4. Type "B" riprap, in place sample Sta. 72+00, 30' Right of "A" line.

Table A-3 Bedding and Spalls Gradation Tests

Sieve Size	Specs Limits % by Weight Passing Spalls	Bedding	Spalls 3 Dec. 1973	Spalls 28 Jan. 1974	Spalls 6 Feb. 1974	Spalls 14 Feb. 1974	Spalls 3 Dec. 1973	Spalls 6 Dec. 1973	Spalls 3 Dec. 1973	Spalls 3 Dec. 1973
7"	100		100	100	100	100	100	100		
5"	75 - 95		92.9	91.3	92.1	94.4	97.5	91.0		
3"	40 - 60		43.3	43.4	47.2	56.6	51.8	42.7		
2"		100							100	100
1 1/2"	4 - 25		14.3	13.6	14.6	17.7	14.5	13.9		
1"		75 - 95							74	84.2
1/2"		35 - 65							27.4	36.6
1/4"		5 - 15							5.4	11.4

Sieve Size	Specs Limits % by Weight Passing Spalls	Bedding	Bedding 4 Dec. 1973	Bedding 6 Dec. 1973	Bedding 4 Feb. 1974	Bedding 18 Feb. 1974	Bedding 19 Feb. 1974	Bedding 19 Feb. 1974	Bedding 20 Feb. 1974	
7"	100									
5"	75 - 95									
3"	40 - 60									
2"		100	100	100	100	100	100	100	100	100
1 1/2"	5 - 25									
1"		75 - 95	91.9	90.7	86.4	90.0	84.2	87.3	84.8	90.8
1/2"		35 - 65	51.6	50.4	39.4	47.2	27.5	35.9	34.1	52.3
1/4"		5 - 15	14.3	9.6	10.6	15.3	6.1	11.3	9.6	14.6

Table A-4 Channel Scalped and Choker Course Gradations

Screen Size	Percent Passing Spec. Limits Channel Scalped	Spec. Limits Channel Choker	Test Number 1	2	3	4
18"	100		100	100		
12"			752	82.7		
10"		100	62.6	64.0	100	100
6"			34.5	30.8		
4"		70 - 95	19.6	15.5	93.4	87.4
2"	10	45 - 75	7.1	6.4	73.2	71.3
-2"		15% pass 200				

1 - Channel Scalped rock in place 185' left of Sta 86+60 on "B" line, El. 663 performed 15 January 1974.

2 - Channel Scalped rock, in place 4' below the shoulder 20' downstream of P. T., right bank.

Table A-4 Channel Scalped and Choker Course Gradations -- con.

3 - Channel Choker course, in place, 5' down from the top of slope, 20' downstream of P.T., right bank.

4 - Channel Choker Course, in place 5' down from the top of slope, 100' downstream of P.T., right bank.

Table A-5 In-place Upstream Slope Protection Gradation Tests

Weight Distribution	Gradation Record Spec. Limits Percent Retained	Test Number (1)	(2)	(3)	(4)
401-1000#	30+%	38.7	33.8	35.7	37.6
201-400#		20.4	17.2	8.1	12.2
101-200#		10.0	15.2	23.0	14.8
11-100#		23.4	26.9	27.0	29.7
0-10#		7.5	6.9	6.2	5.7

1. In place gradation Station 50+50, El. 690, 10:1 slope, 3 June 1974.

2. In place gradation Station 36+44, El. 716.5, 207' U/S, 24 Oct 1973.

3. In place gradation Station 33+00, El. 713, 237' U/S, 10 Oct 1973.

4. In place gradation Station 32+00, El. 715, 10:1 slope, 28 Aug 1973.

5, 6, 7. Permenant slope protection stockpile, 21 June 1973.

Table A-6 Gradation of Upstream Slope Protection Material From Downstream Slope Protection Stockpile

Weight Distribution	Gradation Record Spec. Limits Percent Retained	Test Number (5)	(6)	(7)	
+1000 lb		14.9	5.0	9.5	
401 - 1000 lb	30+%	43.6	26.8	20.3	
201 - 400 lb		14.4	10.8	19.1	(Stockpile
51 - 200 lb		18.4	30.8	32.1	reworked
10 - 50 lb		2.7	21.1	13.0	for finer
1 - 9 lb	5 - 15%	6.0	5.5	6.0	gradation.)
DATE:		20 - 21 June 72			

Table A-7 Average Weight of Upstream Slope Protection Stone

Screen Size	Test Number 1	2	3	4
24"	948-19.2	683-53.0	897-34.7	1010-23.0
18"	493-43.2	311-9.4	477-27.7	365-31.6
12"	196-17.8	195-17.8	170-23.1	138-25.2
6"	38-13.6	38-13.6	46-8.0	57-14.8
5"	8-1.5	8-1.5	9-0.6	7-0.8
(Date)	26-10-71	26-10-71	27-8-71	17-8-71

	Test Number 5	6	7
24"	906-11.0	755-4.9	408-4.7
18"	436-31.7	538-12.1	556-11.4
6"	149-19.8	16.4-18-4	186-19.7
5"	29-25.4	33-31.1	30.7
-5"	9-2.9	11-8.5	6.5
(Pete)	17-8-71	16-8-71	16-8-71

Table A-8 Gradation Tests Choker Course Zone

Screen Size	Percent Passing Spec Limits	Test Number 1	2	3	4	5	6	7	8	9
10"	100	100	100	100	100	100	100			100
8"		100	100		98.7		100			100
6"		96.2	94.6	96.8	93.0	98.8	95.3	100	100	97.6
4"	75 - 95	76.4	79.7	80.9	74.3	88.4	82.3	95.8	96.6	92.9
2"	45 - 75	45.1	58.1	45.7	42.6	64.5	50.8	74.6	68.6	72.1
-2"										
#200	0 - 15	13.0	12.1	10.5	10.1	15.3	9.7	8.4	13.7	6.9

Screen Size	Test Number 10	11	12	13	14	15	16
10"	100	100	100	100	100	100	100
8"	100	100	100	100	97.4	100	96.3
6"	98.5	94.6	100	100	96.5	96.4	95.3
4"	88.9	84.2	94.7	93.6	88.0	80.2	83.4
2"	56.5	59.7	77.9	64.3	64.3	62.1	62.1
-2"							
#200	11.6	14.7	16.0		14.8	16.1	16.1

1 - In place Sta. 47+70, 60 D/S, El. 672, 31 Oct 1973

2 - Belt sample 6 Nov. 1973

Table A-8 Gradation Tests Choker Course Zone -- con.

3 - Belt sample 26 Sept. 1973

4 - In place Sta. 38+50, 75 D/S, El. 710 27 Aug 1973

5 - Belt sample 21 Sept. 1973

6 - Belt sample 12 Sept. 1973

7,8 - In place sample downstream of left nonoverflow bulkhead, 7 Aug 1973

9 - In place sample downstream of left nonoverflow bulkhead, 20 July 1973

10 - Cable tunnel 10 May 1973

11-15 - Choker Course stockpile 8 May 1972, 25 April 1972, 26 April 1972

16 - 19 April 1972, 31 April 1972, 4 April 1972

Table A-9 Gradation Tests of Scalped Rock Zone

Screen Size	Specs	Percent Passing Test Number 1	2	3	4	5	6	7	8	9	10	11	12	13
18"	100	100	100	100	100	100	100	100	100	100	97.2	93.4	96.0	95.8
12"		69.6	95.6	82.4	86.9	54.7	73.5	82.4	69.4	63.7	78.8	66.8	70.8	68.7
6"		30.0	28.4	37.2	47.3	18.5	36.1	37.2	7.4	8.5	10.6	15.4	9.0	7.7
4"		15.4	8.8	12.3	21.7	10.8	20.3	12.3	4.2	4.6	5.7	4.4	3.5	3.2
2"	0-10	6.3	4.8	5.4	9.3	4.6	9.4	5.4	2.4	2.8	3.4	2.4	2.3	1.7
-2"														

1. In place sample, Station 47+75, 50 D/S, El. 695, 14 June 1974.

2. In place sample, Sta. 47+50, 40 U/S, El. 664, 26 September 1973.

3. In place sample, Station 38+57, 107 D/S, El. 711, 27 August 1973.

4. In place sample, Sta. 38+50, El. 711, 28 August 1973.

5. In place sample, Sta. 38+60, 180 D/S, 30 July 1973.

6. In place sample, 27 July 1973.

7. In place sample, Sta. 38+51, 107 D/S, 30 July 1973.

8, 9, 10, 11, 12, 13. Stockpile sample 28 July 1972, 14 July 1972, 30 June 1972, 26 April 1972, 20 April 1972, 24 March 1972.

Table A-10 Gradation Tests Rock Zone I

Screen Size	Per Cent Passing Test Number 1	2	3	4	5	6	7	8
24"				96.5				100
18"	100	100	100	89.8	100		100	98.5
12"	96.7	96.8	97.4	84.4	99.6	100	98.9	96.4
6"	83.9	84.2	88.5	70.3	91.2	95.1	93.6	88.8
4"	71.6	75.4	78.8	64.5	83.3	84.9	88.2	81.2
2"	53.3	58.1	57.2	54.9	70.7	66.3	72.9	64.5

1 - 31 May 1974, Sta. 50+25, 225' u/s, El. 694

2 - 30 May 1974, Sta. 47+85, 110' u/s, El. 693

3 - 7 May 1974, tailrace excavation, Zone 1 rock

4 - 3 October 1973, Sta. 48+00, 120' u/s, El. 662

5 - 16 October 1973, Sta. 38+30, 65' u/s, El. 705

6 - 13 August 1973, excavation

7 - 2 August 1973, Zone 1 rock excavation

8 - 14 July 1973, Zone 1 rock excavation, Sta. 50+00, Range 2+00 L.

Table A-11 Gradation Tests Rock Zone 2

Screen Size	Per Cent Passing Test Number 1	2	3	4	5	6	7	8
36"	100	100	100	100	100			100
24"	94	93.4	96.1	85.4	96.6	96.8	100	85.7
18"	90.9	93.4	96.1	84.5	95.8	96.8	97.5	
12"	86.6	84.9	90.1	80.0	89.1	94.2	88.4	76.8
6"	77.0	74.5	81.4	64.0	66.8	85.2	77.1	58.1
4"	69.1	64.2	71.2	54.1	54.3	75.0	69.9	48.3
2"	54.8	45.8	54.2	41.1	39.0	54.6	54.7	34.7

1 - 1 June 1974, In place gradation, Sta. 47+50, 40' D/S

2 - 29 May 1974, In place gradation, Sta. 47+60, 60' U/S

3 - 10 Oct 1973, In place gradation, Sta. 47+31, 257' D/S, El. 661

4 - 2 Aug. 1973, Stockpile sample

Table A-11 Gradation Tests Rock Zone 2 -- con.

5 - 28 July 1973, Spillway excavation

6 - 14 July 1973, required excavation

7 - 15 June 1973, required excavation, Sta. 54+80, Range 2+38R
El. 641.42 - 651.8, Spring 5 1/2 x 11

8 - 18 May 1973, required excavation, Sta. 47+35, Range 1+30R
El. 640 - 652, Spring 5 x 10

SUPPLEMENT B

ROCK DENSITY TESTS

1-01. Introduction: In design of a dam, knowledge is necessary on all consitutents of the structure; whether it is the characteristics of the impervious core, the random sections, rock sections or pervious drains. Physical properties of soils can, in most cases, be learned through labortory testing; but rock in many cases is too large and unwieldy to afford the conventional tests of a laboratory. At Harry S. Truman Dam, a rock density test was developed primarily by trial, error, and study, which provides a compacted unit weight with a reasonable amount of accuracy. The following narrative explains the history of development, test by test, through the Stage II contract and the results of an acceptable approach applied in the Stage III contract.

1-02. Rock Density Tests, Stage II: The tests performed under the Stage II contract could be defined as experimental in conjunction with a learning period. Each test is described individually in detail in search for an appropriate standard procedure. As can be determined, the results of the four tests were erratic with questionable validity. An improvement in the test procedure was finally developed and applied to the Stage III contract. The following table shows in summary the relationship of the test results between Stage II and Stage III contracts.

Table B-1

Compacted Unit Weights

	Stage II	Stage III
Sample 1	135.5 #/CF	127.0 #/CF
Sample 2	124.2 #/CF	130.2 #/CF
Sample 3	158.0 #/CF	127.6 #/CF
Samp' '	118.7 #/CF	131.6 #/'CF

1-03. Rock Density Tests, Stage III:

a. Procedures: An improvement in the test procedures was adopted to subscribe to more valid results than were obtained during Stage I. Instead of a certain weighted sample, a fixed volume of rock was used for each test. The dimensions of the sample were the same for each test. In these cases, sample thickness coincided with lift thickness; hence the compactive effort should have been uniform throughout the test samples. Two factors attesting to the improvement of the test are the sample weight and the resultant compacted unit weight. Sample weights were reasonably uniform and the resultant unit weights of the four tests ranged within five pounds per cubic foot of each other. The most recent test method adopted is as follows: The sample area was measured as 8 feet wide and 12 feet long. Sample thickness measured was 3 feet coinciding with the thickness of the rock lift. A rock gradation was made of the sample removed within the dimensions 8 x 12 x 3. This was a rock

1-06. Rock Density Test Number Three: Rock Zone 1, Elevation 689.5 - 691.5 Sta. 54+40, Range 228.4 Upstream.

a. Procedures: On 27 September 1968, a 11.61 ton sample of rock was removed from Shot 17 of 2nd lift, Area B and graded according to paragraph 9.1 of the specifications entitled Rock Gradation Tests, Rock Fill and Slope Protection. During the initial grading, the sample was delivered to the hole and placed by hand. The intent was to minimize the loss incurred through grading the hole (attempted in Density Test 2) since the total sample weight could not be obtained until the total sample was already placed. Efforts nevertheless were directed toward achieving control and a condition reflecting the embankment in place. Only reasonable success has generally been achieved in instituting a representative sample with the correct lift thickness. A total of four survey sections were made during the testing operation. Prior to introducing the sample into the hole, a polyethylene liner was installed and the initial survey section determined the hole volume. The second section was made atop the sample in place before compaction. The third section was taken after sample compaction and a final section was made of the hole after the sample removal was completed. The findings resulting from these surveys provided the information to determine the unit weight of the sample before and after rolling and the amount of consolidation. The final feature requirement of the test, the measure of the amount of breakdown, was determined by comparing gradations before and after rolling of the sample was accomplished. The essence of the test was to grade the sample, roll the sample in place (four passes of the Fergerson Vibrator Model 230-working weight of 23,5600 lbs.) and regrade the sample.

b. Results: Comparing the gradations taken, the sample after rolling reflected a particle breakdown of 1.3 percent to 4.8 percent. Other data secured during the performance of this test is listed below:

Volume of Sample Uncompacted	196.9 CF
Volume of Sample Compacted	142.8 CF
Loss Through Compaction	27.5%
Unit Weight of Sample Before Compaction	117.9 lbs./CF
Unit Weight of the Rock After Compaction	158.0 lbs./CF

Sample Weight

23,217 Lbs.	Gradation 1
22,562 Lbs.	Gradation 2
655 Lbs.	

Rock Density Test #2

Figure B-2

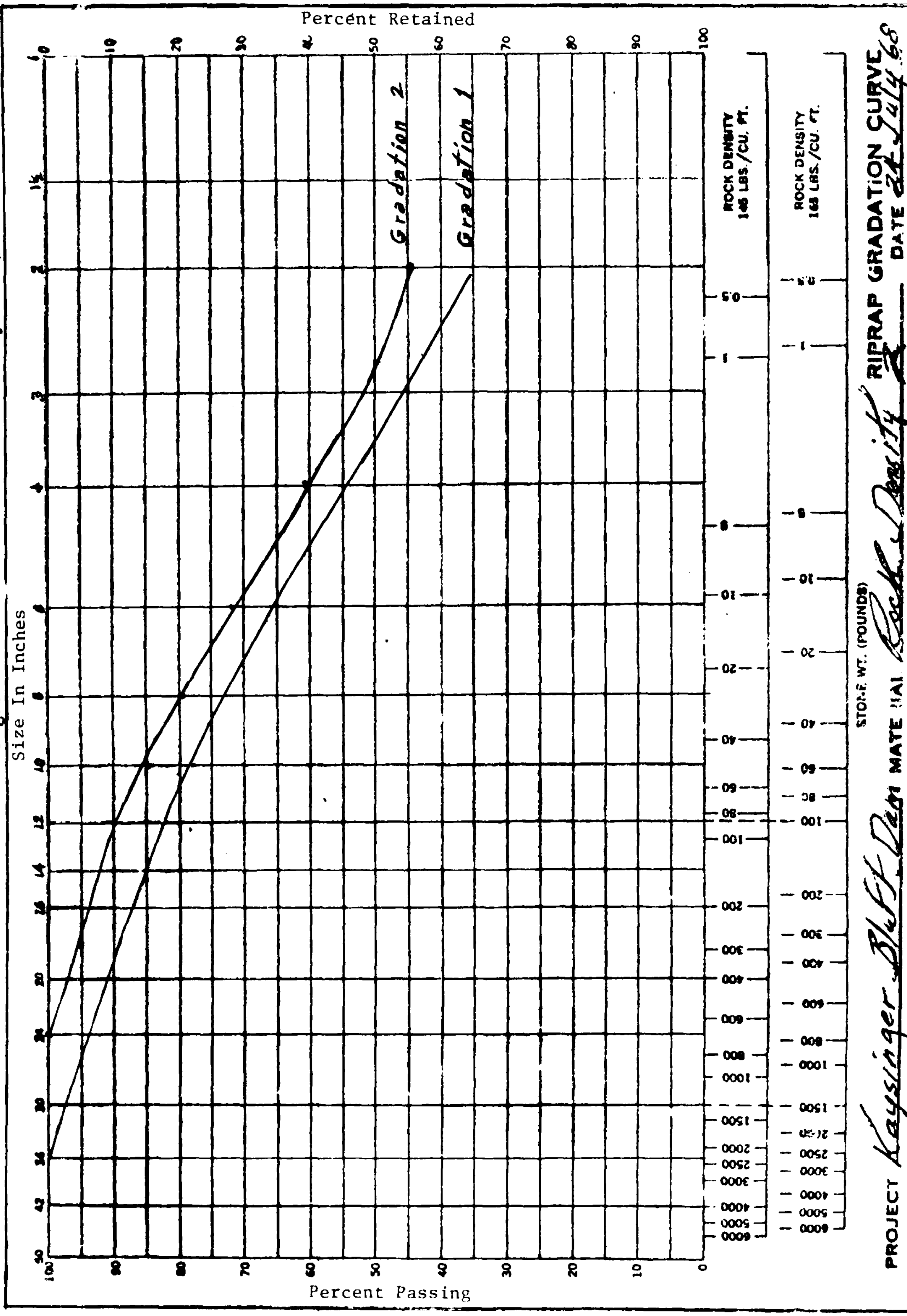

Figure B-2

B-10. Regrading sample after compacting.
16 July 1968, Neg. No. 1026

c. Conclusions: Reference is made to Rock Density Test 1, Rock Zone 2, Horizontal Drain. The problems that directly affected an accurate evaluation of the results in Density Test 1 were reduced by the changes made in the running of Density Test 2. The unit weight of the rock is 10.4 lbs./CF less than that obtained in Rock Density Test 1. This reduction could be attributed to either one or all of the following factors: (1) Inaccuracies in surveys. (2) Different method of compacting the sample in each test. (3) Different inplace sample thickness in each test. Sample thickness in Density Test 2 was roughly twice that of Density Test 1. One refinement which will be implemented on future tests is the reduction of sample loss. Loss in handling the sample of Density Test 1 was 0.59 ton whereas loss in handling the sample of Density Test 2 was 0.52.

/S/
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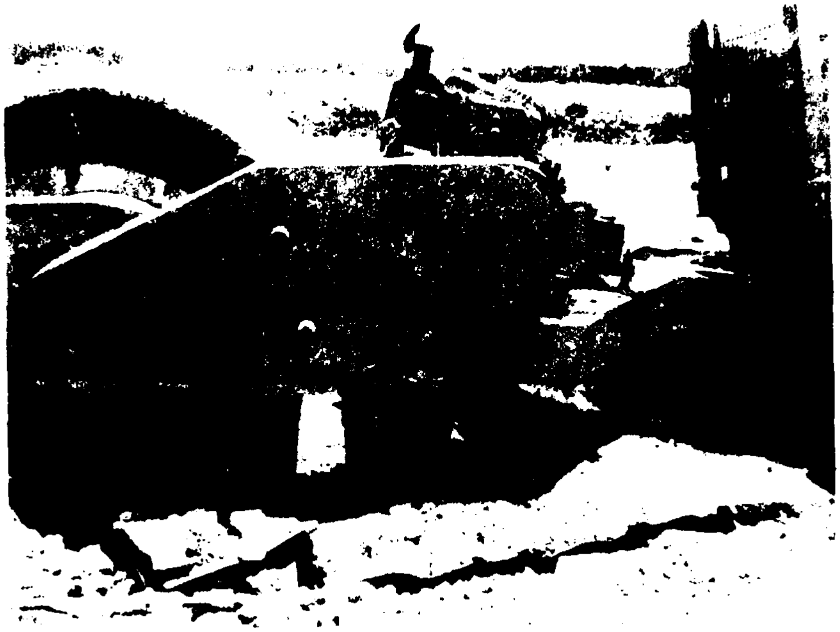
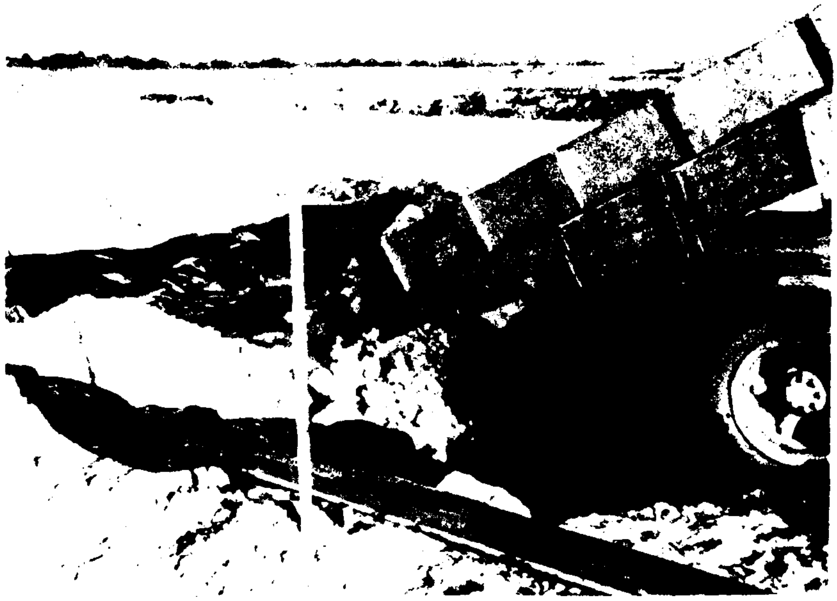


B-8. Rolling sample with vibratory roller.
11 July 1968, Neg. No. 1020-15

B-9. Sample surface after rolling.
11 July 1968, Neg. No. 1020-13



B-6. Dropping sample into hole.
11 July 1968, Neg. No. 1020-16

B-7. Sample in place before rolling.
11 July 1968, Neg. No. 1020-14

Table B-5
Rock Density Test 2
Record of Gradations

	GRADATION 1			GRADATION 2			
Screen Size	Weights	Individual Percentages	Percent Passing	Weights	Individual Percentages	Percent Passing	Percen Break down
36	853	1.9	100	0	0.0	199	0.0
30	1,845	4.2	98.1	0	0.0	199	1.9
24	2,374	5.4	93.9	2,080	4.8	100	6.1
18	2,488	5.6	88.5	2,302	5.4	95.2	6.7
12	2,347	5.3	82.9	2,173	5.1	89.8	6.9
10	2,577	5.8	77.6	2,550	5.9	84.7	7.1
8	3,307	7.5	71.8	2,801	6.5	78.8	7.0
6	4,696	10.7	64.3	4,644	10.8	72.3	8.0
4	7,549	17.1	53.6	7,475	17.4	61.5	7.9
2	16,031	36.5	36.5	18,950	44.1	44.1	7.6
	44,067			42,975			

Unit Weight 124.15 lbs./CF

Sample Weight

22.00 Ton Gradation 1
21.48 Ton Gradation 2
0.52 Ton Weight Loss

1-05. Rock Density Test Number Two: Rock Zone 1, Elevation 679-682 Sta. 58+50, Range 107' Downstream.

a. Procedures References: Paragraph 8 of the Specifications for Cosntruction of Kaysinger Bluff Dam, Stage II Construction; entitled "Rock Density Tests". On 1 July 1968, a 22.0 ton sample was removed from shot 5, lift 1, Area B, and graded according to paragraph 9.1 of the specifications entitled Rock Gradation Tests, Rock fill and Slope Protection. Based on the weight of the sample graded, a hypothetical volume was calculated from the unit weight of the rock obtained from Rock Density Test Table B-5, Gradation 1.

$$\text{Vol. Hyp.} = \frac{44{,}067}{135.5 \text{ lbs./CF}} = 325.2 \text{ CF}$$

The Contractor located a sector 10' x 10' in rock zone one and directed the rock placement operation around this sector in order to provide a hole 10' x 10' in the 3 foot lift being placed. The hole was finally shaped by hand labor. Because of some inaccuracies in achieving the desired results in the field, the hole assumed a volume and dimension different from that which was calculated (design volume of 325.2 CF vs. actual volume of 497.88 CF. Prior to introducing the sample into the hole, a polyethylene liner was installed and a survey was made to determine the volume of the hole by level rod readings. On 11 July 1968, the sample was carefully loaded by hand shoveling and end loader onto a flat bed stake truck for delivery to the hole already located and prepared. Six loads of samples were delivered to the site. By this method, segregation was held to a minimum and breakage was reduced by the short drop from truck to hole (Reference Photo B-6). The sample was levelled in the hole by hand labor and another survey was made atop the sample before compaction (Reference Photo B-7). Compaction was applied by a Ferguson Vibrator Model 230 (working weight of 23,500 lbs.) towed by a 977 Model Tracked Caterpillar end loader (Reference Photo B-8). The equipment traversed the sample in conjunction with rolling the surrounding lift until four passes were completed. Before the sample was disturbed, another top of rock survey was made to reflect the amount of consolidation (Reference Photo B-9). After the necessary surveys were accomplished the sample was removed by hand and regraded to determine the amount of particle breakdown. See Photo B-10, and Table B-5, Gradation 2. To ascertain changes in the hole by introducing the sample, rolling it, and removing it; a final survey was made after the sample removal was completed.

b. Results: The sample experienced a breakdown ranging from 2 to 8 percent with a relatively uniform breakdown varying not more than 2 percent from the 24 inch screen to minus 2 inches, Table B-5. The final survey data produced the following figures.

Original Volume of the Hole		18.44 c.y.
Final Volume of the Hole		18.46
Volume of the Sample Uncompacted		15.7 c.y.
Volume of the Sample Compacted		12.8 c.y.
Unit Weight of the Rock after Compaction		124.15 lbs./CF
Sample Weight		
Gradation 1	22.00 Tons	
Gradation 2	21.48 Tons	
Difference	0.52 Tons	

Figure B-1

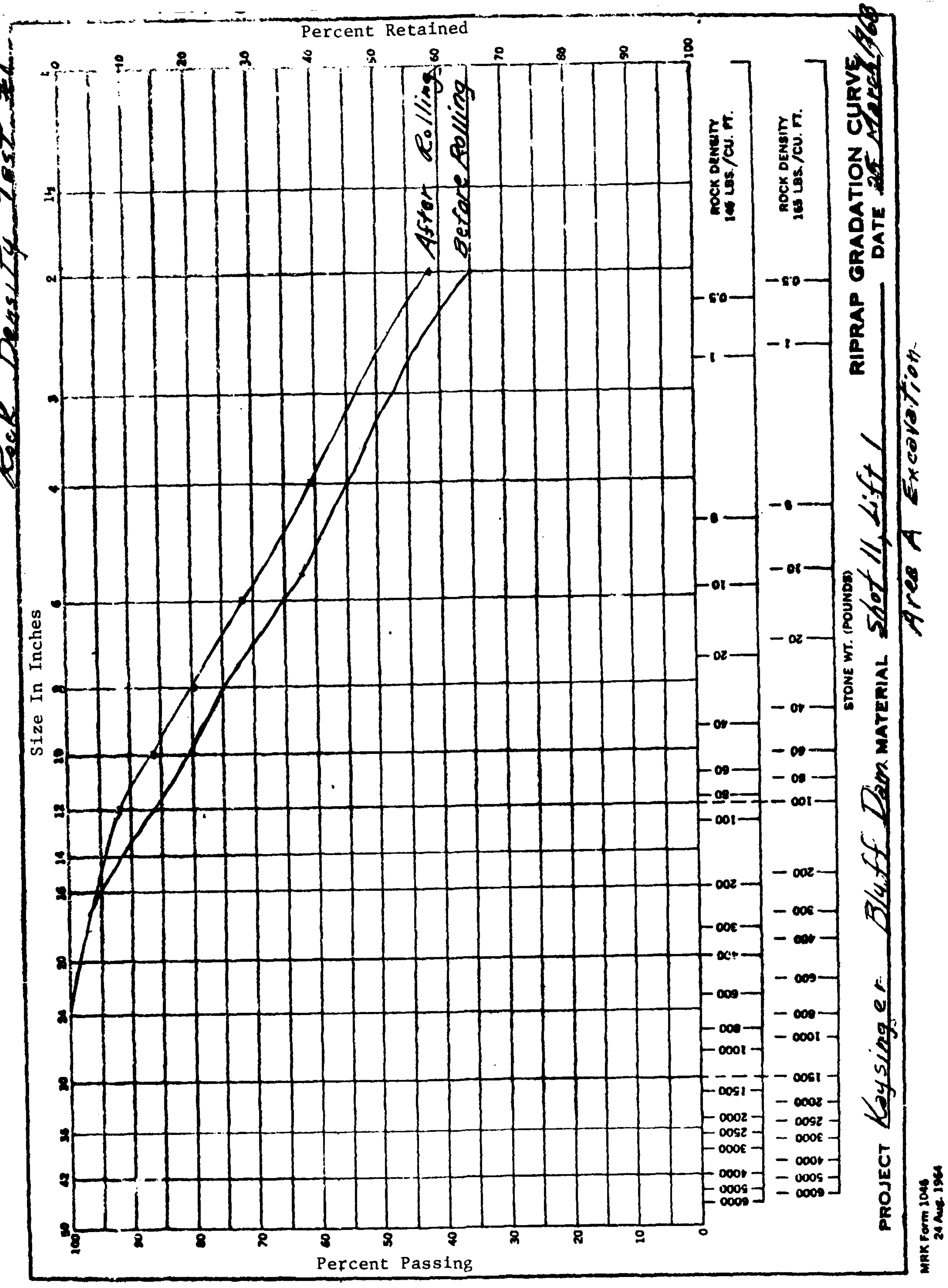


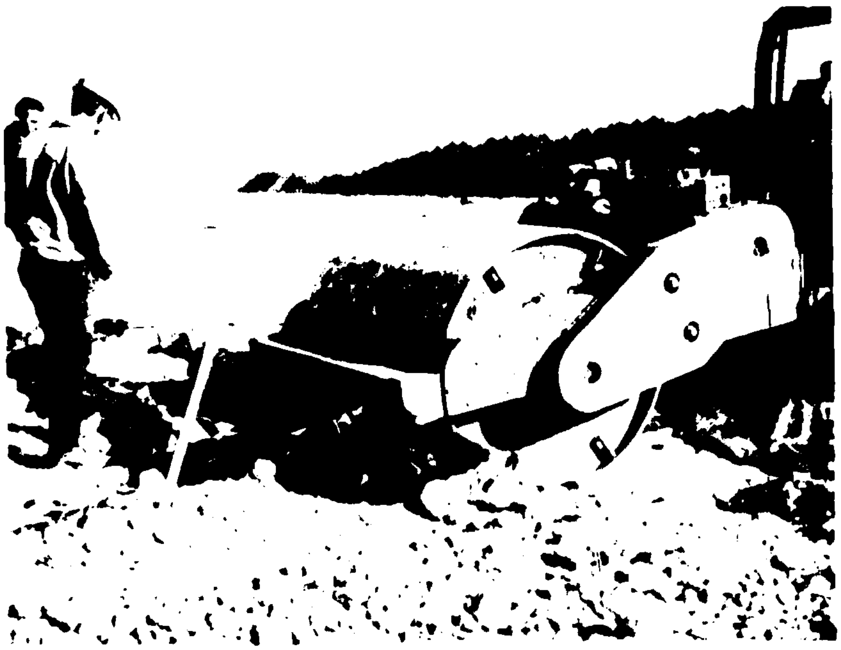
Figure B-1

B-4. Sample surface after four passes of vibrater roller. 25 March 1968, Neg. No. 85144.

B-5. Regrading sample after compacting. 26 March 1968, Neg. No. 85150-A

ROCK DENSITY TEST NO. 1

B-2. Leveling sample surface.
23 March 1968, Neg. No. 8514[illegible]



B-3. Rolling sample with [illegible]
23 March 1968, [illegible]

In an effort to improve on the testing technique a proposed new method was submitted to Kansas City District Office on 11 April 1968. Comments from this proposal and improvements on the testing methods in the field intend to refine the Rock Density Test to a higher level of authenticity.

/S/
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ROCK DENSITY TEST NO. 1

B-1. End dumping sample into hole.
23 March 1968, Neg. No. 85141.

a. Procedures: On March 18, 1968, an 8.75 ton sample of rock was removed from shot number 11, lift 1, Area A excavation, and graded according to paragrapn 9.1 of the Specifications entitled Rock Gradation Tests, Rock Fill and Slope Protection. On March 23, 1968, the sample was loaded, by hand and with use or a front end loader, onto a flat bed stake truck, for delivery to the hole pinpointed and excavated earlier. The hole had been excavated with front end loader and lined with polyethylene to separate the country rock from the sample. An initial survey section was made by level rod readings to determine the volume of the hole at this time. After surveys were completed, the rock was carefully dropped into place. See Photo B-1. Plywood mats were used to reduce rock on rock impact and prevent punctures in the polyethylene liner. After placement, the rock mass was levelled to permit maximum bearing of the vibratory roller on all parts of the mass. See Photo B-2. In levelling the sample, it was noted that the majority of the fines was prominently located near the surface. This condition was created during the course of end dumping the sample off the flat bottom truck. Segregation occurred permitting the coarser rock to drop into the hole first. On March 25, 1968, the sample was rolled with four passes of the vibratory roller. The roller was backed over the sample and pulled forward for two complete cycles. See Photo B-3. Though a survey section of the sample surface in the hole was not obtained before rolling, sectioning was accomplished after rolling and before the sample was removed. See Photo B-4. After the necessary surveys were accomplished the sample was removed by hand and regraded to determine the amount of particle breakdown. See Photo B-5. To determine volume changes of the hole before the sample was introduced and after the sample was removed a final section was made of the hole after removal was completed.

b. Results: The sample experienced a 5 to 7 percent breakdown below the 18 inch screen. (See Gradation Curve, figure B-1). The following data was extrapolated from the survey work performed in conjunciton with the test.

Original Volume of the Hole	20.80 c.y.
Volume of Compacted Sample	4.46 c.y.
Volume of Hole After Sample Removal	18.27 c.y.
Final Weight of Sample	8.16 ton
Unit Weight of Rock After Compaction	135.5

c. Conclusions: Rock Density Test Number One was performed as stated in the specifications. Problems affecting accuracy of results were:

1. Fitting the sample to match the hole to achieve a uniform compactive effect applied during a normal rolling operation.

2. Affecting the sample thickness to match that of the lift tested.

3. Developing an accurate method of determining volume using level rod readings.

Table B-3

Comparison of Density Tests

	(1) Accuracy	(2) Volume Change	Compacted Unit Weight
Sample 1	98.1%	8.2%	127.0 #/CF
Sample 2	97.4%	18.2%	130.2 #/CF
Sample 3	99.2%	11.4%	127.6 #/CF
Sample 4	99.8%	16.6%	131.6 #/CF

Notes: (1) Accuracy is an expression of the variance in total weights of the sample graded twice. Variance expected to be caused by human error in weighing, or inadvertent gaining or losing of some of the rock.

(2) Volume change is shrinkage of the sample hole from rolling.

Table B-4

Rock Gradations Before and After Compaction

	% Passing	36"	24"	18"	12"	6"	4"	2"
Sample 1	Before Rolling	100	98.3	93.5	90.3	83.2	77.0	63.6
	After Rolling	100	98.3	93.8	91.7	83.9	78.7	65.5
Sample 2	Before Rolling	96.6	94.2	93.0	89.7	82.7	75.5	64.5
	After Rolling	100	94.2	90.7	89.7	81.8	75.5	64.9
Sample 3	Before Rolling	91.0		85.1	76.2	63.4	56.1	42.7
	After Rolling	90.9		86.2	80.0	66.0	57.6	44.4
Sample 4	Before Rolling		94.4	94.4	91.2	82.9	76.3	63.4
	After Rolling		94.4	94.4	91.4	83.8	78.6	65.2

c. Conclusions: Four features were determined during the course of performing the rock density test. The first feature was the use of this test as a test pit. Examination of the walls and floor of the hole were accomplished prior to rolling the sample and after the sample and country rock were rolled. The second feature was a comparison of gradation results. The comparison provided some insight on the amount of rock breakdown attributed to rolling. The third feature was a comparison of hold size before and after the area (plus sample) was rolled. The fourth feature was the comparison of the rock's unit weight before and after rolling. It is believed that since there is now a measure of uniformity in the test samples, a relationship between the tests may be ascertained. A relationship could help provide character to the test and justification for performing it.

1-04. Rock Density Test Number One: Rock Zone 2, horizontal drain, Sta 59+16, Range 216 Downstream.

gradation of the in-place sample before rolling. The hole remaining was sectioned by the Contractor's survey party. A proyethylene liner ws fitted intc the hole to separate the sample from the surrounding rock. The sample was reintroduced uniformly into the hole. Survey sections were again obtained. The area was rolled with four passes of the Model VP20D Bros. vibrating roller. The sample was again removed for another rock gradation. This was a rock gradation of the in-place sample after rolling. Final survey sections were taken of the hole after the sample was removed.

b. Results: The following tables offer a comparison of the four density tests pervormed as described hereinbefore. Usable information obtained from these tests for design purposes would be to the discretion of the designer.

Table B-2

Rock Density Tests

Sample 1 13 August 73 Sta. 37+80 Range Not Recorded El. 705

Rock Zone 2	Sample Weight	Sample Volume	Unit Weight
Before Compaction	73,720 lb.	11.76 cy or 317.52 CF	118.80 #/CF
After Compaction	36,990 lb.	10.79 cy or 219.33 CF	126.97 #/CF
	98.1%		

Sample 2 27 September 73 Sta. 47+80 310' U/S El. 665

Rock Zone 1	Sample Weight	Sample Volume	Unit Weight
Before Compaction	36,719 lb.	13.11 cy or 353.97 CF	103.73 #/CF
After Compaction	37,696 lb.	10.72 cy or 289.44 CF	130.24 #/CF
	97.4%		

Sample 3 16 November 73 Sta. 49+67 230' D/S El. 675

Rock Zone 2	Sample Weight	Sample Volume	Unit Weight
Before Compaction	41,437 lb.	13.47 cy or 363.69 CF	113.93 #/CF
After Compaction	41,098 lb.	11.93 cy or 322.11 CF	127.59 #/CF
	99.2%		

Sample 4 10 December 73 Sta. 48+90 200' U/S El. 676

Rock Zone 1	Sample Weight	Sample Volume	Unit Weight
Before Compaction	30,501 lb.	10.27 cy or 277.29 CF	110.00 #/CF
After Compaction	30,451 lb.	8.57 cy or 231.39 CF	131/6 #/CF
	99.8%		

Table B-6
Rock Density Test 3
Record of Gradations

	GRADATION 1			GRADATION 2			
Screen Size	Weights	Individual Percentages	Percent Passing	Weights	Individual Percentages	Percent Passing	Percent Breakdown
36	0	0.0	100.0	0	0.0	100.0	0.0
30	0	0.0	100.1	0	0.0	100.0	0.0
24	1,246	5.4	100.0	940	4.1	100.0	0.0
18	2,517	10.9	94.6	1,696	7.5	95.9	1.3
12	723	3.1	83.7	807	3.6	88.4	4.7
10	1,378	5.9	80.6	1,190	5.3	84.8	4.2
8	1,562	6.7	74.7	1,832	8.1	79.5	4.8
6	2,135	9.2	68.0	2,115	9.4	71.4	3.4
4	3,903	16.8	58.8	3,695	17.6	62.0	3.2
2	9,753	42.0	42.0	10,017	44.4	44.1	2.4

Unit Weight

Sample Weight

11.61 Ton Gradation 1
11.28 Ton Gradation 2
0.33 Ton Weight Loss

c. Conclusion: Subject to closer examination is the wide range in Unit Weights obtained from these tests.

/S/
JOHN W. DOTY
Geologist

ROCK DENSITY TEST NO. 3

B-11. Sample placed in hole before rolling.
27 September 1968, Neg. No. 1020-12

B-12. Sample being compacted with roller.
30 September 1968, Neg. No. 1020-11

B-13. Sample beign compacted with roller.
30 September 1968, Neg. No. 1020-10

B-14. Sample after rolling.
30 September 1968, Neg. No. 1020-12A

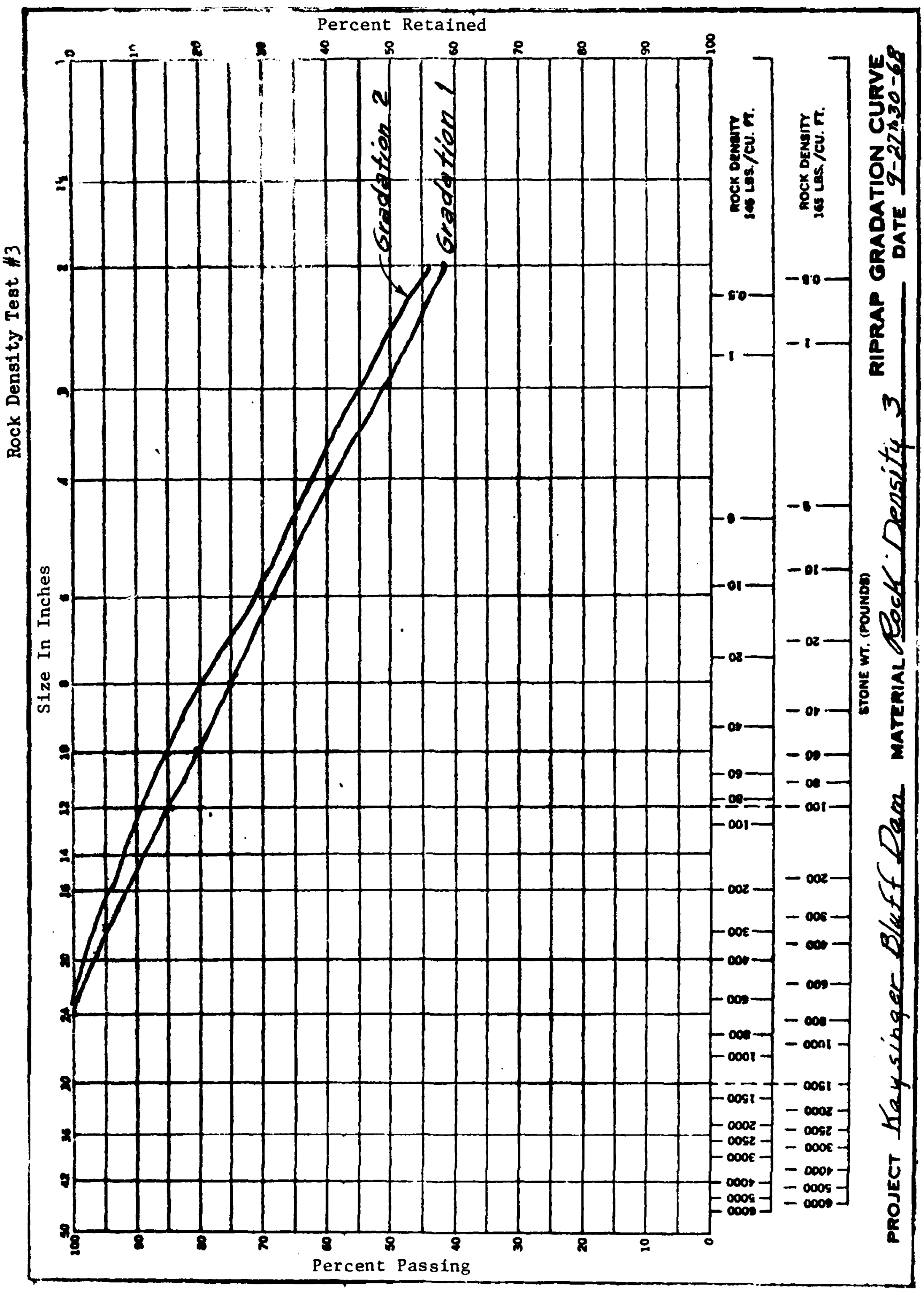

Figure B-3

Figure B-3

1-07. Rock Density Test Number Four: Rock Zone 1, Elevation 7-6.0 - 709.0

a. Procedures: On 31 October 1968, 7.2 Ton sample of rock was obtained from the byproduct stockpile formed through the manufacture of Upstream Slope Protection; and graded according to paragraph 9.1 of the specifications entitled Rock Gradation Tests, Rock Fill and Slope Protection. This material was chosen for these reasons. This rock byproduct is ear-marked for incorporation into the dam in Zone 1 above El.701.5 and downstream of the rock raked zone. Thus a measure of importance has been assigned to such factors as, unit weight of the rock in place, comparison of this byproduct gradation with choker course manfactured in a similar manner and gradation of this rock in place after compaction to determine breakdown of smaller (less than 8") rock as separated from the general rock mass. After placement, to satisfy the compaction requirements, four passes of the Ferguson Vibrator Model 230 (23,500 lbs. vibrating roller) was made. See Photo B-16. The sample was then removed and graded again to determine particle breakdown through rolling. The Contractor, in planning the mechanics of this test, excavated the sample hole before the sample was weighed in order to place the sample in the hole during the grading process. See Photo B-15. This approach inadvertently eliminated the hypothetical exercise in calculation the hole size beforehand by the sample weight and unit weight extrapalated from the previous tests. Thus, it was decided to fill the hole as planned except for one condition. In filling the hole the work was concentrated on providing a three foot lift even if only a portion of the hole was used. After completion of the first gradation, the unused portion of the hole was thereby backfilled up to the sample. The sample was already contained by the polyethylene liner which also helped to prevent contamination by the surrounding and backfill rock. Survey sections were made before sample material was placed in the hole; after the sample was placed and backfilled as described, but before rolling; and after the rolling was completed. The final section was made after the sample was removed from the hole. This section provided the esential figures in calculating the volume of the sample since the first survey section held little value after backfilling against the sample was decided. Considerable merit has been given to this approach to the test because the sample can be situated to represent the full lift thickness, and more control helps to assure a more realistic test when the rock is hand placed as opposed to dumping. One step in handling the sample is eliminated by placing the rock in the hole during the gradation operation. This approach reduces rock breadage and sample loss through handling. The primary disadvantage encountered by this approach was the nature of the contact between the sample and the backfilled section of the hole. The sample achieved an angle of repose which in essence ws an obtuse or reverse angle in respect to the backfilled side of the contact. Error was reqlized in trying to obtain a survey section along this contact line. When the sample was removed along the contact, it was difficult to prevent the rock backfill from spilling into the sample sector. See Photo B-17. This, it is believed, accounted for the increased sample weight of Gradation 2 as shown in Table B-7.

b. Results: The rock experienced a particle breakdown from 1.5 to 5.0 percent between 6" and 2" (Table B-7). The final survey produced the following data.

Volume of the Sample Uncompacted 5.03 c.y.
Volume of the Sample Compacted 4.50 c.y.
Loss Through Compaction 10.5 Percent
Unit Weight of Sample Before Compaction 106.2 Lbs./CF
Unit Weight of Sample After Compaction 118.74 Lbs./CF
Sample Weight
14,411 Gradation 1
15,237 Gradation 2
826 lbs. Gain in Weight

Table B-7
Rock Density Test 4
Record of Gradations

	GRADATION 1			GRADATION 2			
Screen Size	Weights	Individual Percentages	Percent Passing	Weights	Individual Percentages	Percent Passing	Percen Break down
8"	1,222	8.5	100.0	1,073	7.0	100.0	0.0
6"	2,830	19.8	91.5	2,700	17.7	93.0	1.5
4"	4,893	33.5	71.9	4,911	32.2	75.2	3.3
2"	5,466	38.2	38.0	6,553	43.0	43.0	5.0
	14,411			15,237			

$$\text{Unit Weight} = \frac{14{,}411 \text{ Lbs.}}{121.37} = 118.74 \text{ Lbs./CF}$$

Sample Weight

14,411 Gradation 1
15,237 Gradation 2
826 lbs. Gain in Weight

Rock Density Test No. 4

B-15. Grading sample before placing in hole. Fill El. 710±.
31 October 1968, Neg. No. 1020-8

B-16. Rolling sample.
31 October 1968, Neg. No. 1020-7

Rock Density Test No. 4

B-17. Grading sample after rolling.
1 November 1968, Neg. No. 1020-5

c. <u>Conclusions:</u> Even though this is the final test in this series of rock density tests, there has been progressive refinement in the approach to taking each test. The approach used in performing this test is a step forward to a practical test procedure that warrants strong consideration. The basic drawback, is the difficulty in measuring and securing the sample along the backfill contact as described herein. Now that the problem has been recognized, it is believed that it can be resolved by field expedients. A final suggestion to aid in overcoming the menial inaccuracies produced during the test, is to specify the minimum weight of the sample as 10 tons or more. another advantage of a sample larger than the 6 to 8 ton sample specified, is the assurance that proper lift thickness can be represented by the sample tested.

/S/
JOHN W. DOTY
Geologist

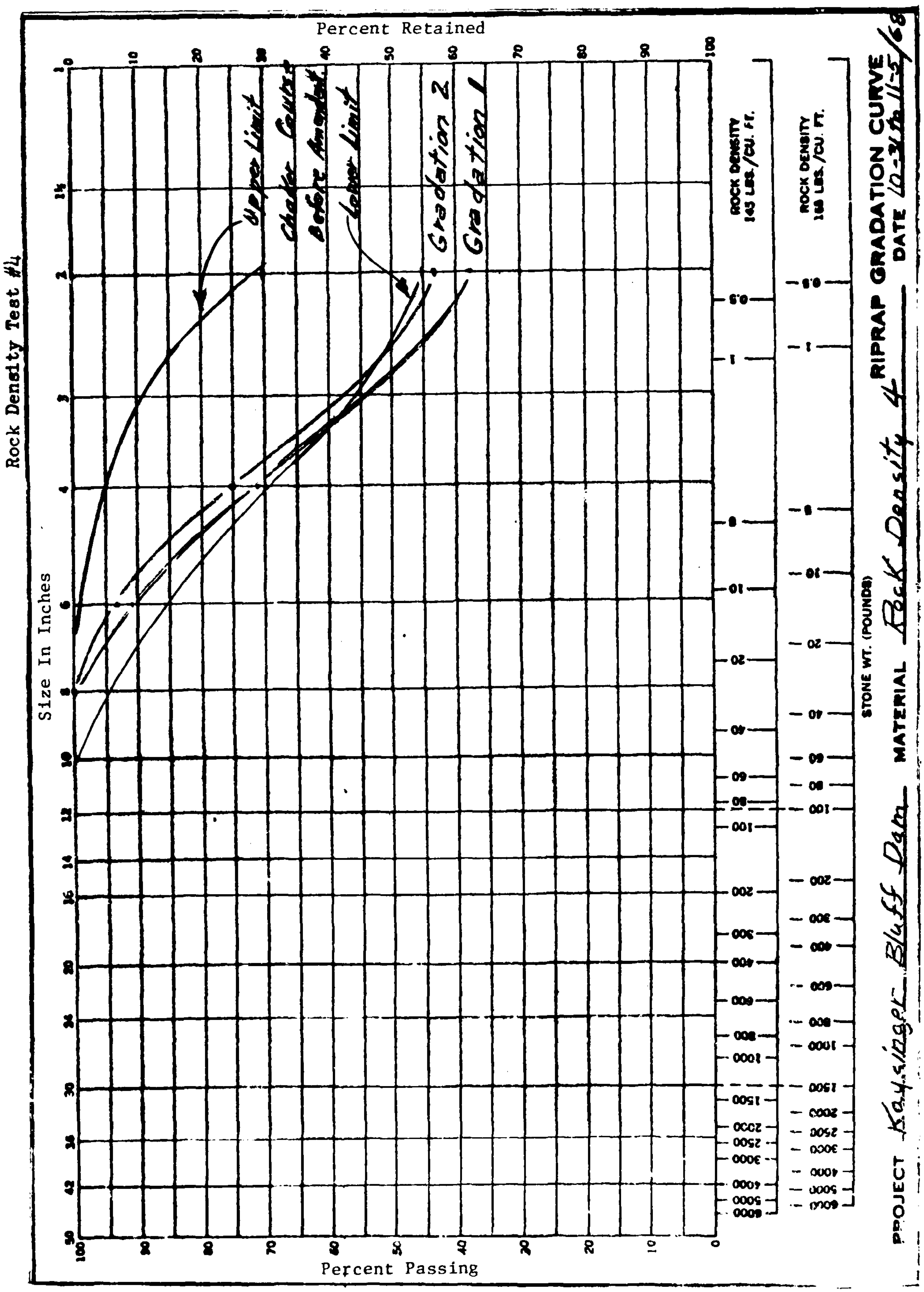
Rock Density Test #4
Percent Retained
Size In Inches
Percent Passing
Upper Limit
Lower Limit
Gradation 2
Gradation 1
ROCK DENSITY 145 LBS./CU. FT.
ROCK DENSITY 165 LBS./CU. FT.
STONE WT. (POUNDS)
RIPRAP GRADATION CURVE
DATE 10-31 to 11-5/68
PROJECT Kaysinger Bluff Dam
MATERIAL Rock Density 4

Figure B-4

SUPPLEMENT C

REPORT ON ROCK EXCAVATION OF THE POWERHOUSE STRUCTURE AREA, STAGE II

1-01. Introduction: In accordance with the requirements of Stage II construction, Western Contracting Corporation drilled and blasted 240,000 ± cubic yards of rock from Area A. Between surface El. 865 ± and floor 816 ±, three lifts were excavated providing bench heights of 20', 15', and 15' for lifts 1, 2, and 3 respectively; (notwithstanding variations in rock creating minor fluctuation in height). To accurately document progress in excavation, refinements in blasting, and to establish blasting guidelines for future work considerable time has been spent in observing each shot. During loading and after each blast, observations were made, noting blast configuration, load, type of explosives, and general fragmentation of the rock. The purpose of this report is to present the principles developed in producing a more satisfactory rock product in accordance with the specifications.

1-02. Rock Specifications and Guidelines: Select portions of the specifications served as a guide in defining and developing a rock blasting plan. The purpose, principles in using explosives, and the range of rock gradations as extracted from the specifications are simply stated herein.

a. Blasting Plan: The purposes for establishing a blasting plan are to produce a maximum amount of well graded material for embankment and to minimize the quantity of rock fines (-2 inch rock).

b. Explosives: The principles governing the use of explosives are (1) Using cartridges (untamped) with a diameter equal to one-half the diameter of the blast hole when loading at a powder factor of approximately 1/2 pound per cubic yard reduces total explosive force and induces decoupling. (2) Using cartridge explosives in preference to bulk type explosives serves as a defense against water, a measure of explosive proportioning and a device assuring decoupling. (3) Subdrilling each hole not greater than two feet serves as a preventative against overdrilling and blasting insitu rock more than once.

c. Gradation: Gradation requirements guiding rock fragmentation are as follows:

1. Scalped rock and channel scalped rock shall have a maximum size - 18 inches, be well graded, and have ten percent or less passing the two-inch screen.

2. Choker course and channel choker course shall have a maximum size of 10 inches, have 70-95 percent passing the 4-inch screen with a 45-70 percent passing the 2-inch screen and not more than 15 percent of the material passing the 2-inch screen shall pass the number 200 sieve. Residual material from the production of scalped rock shall be utilized to the maximum extent possible in the production of material for choker course. On 8 July 1968, a modification was written changing the choker course gradation to read: "45-80 percent passing the 2-inch screen and not more than 22 percent of the material passing the 2 inch shall pass the number 200 sieve when washed."

3. Upstream rock slope protection shall have a maximum size of 1,000 pounds. Thirty percent shall be heavier than 400 pounds, 5-15 percent lighter than 10 pounds, and be reasonably well graded. Loads of more durable material shall be selected from Area "A" excavation.

4. Downstream rock slope protection shall be well graded rock selectively loaded from Area "A" excavation, rock quality to be determined by the Contracting Officer.

5. Rock Zone 2 - horizontal drain shall be rock selectively loaded from Area "A" containing at least 20 percent passing 2-inch screen prior to compaction.

d. Rock Utilization: A maximum utilization of all the rock was imperative to attain the proper balance of quantities between excavation and embankment set forth in the project design. To attain a perspective in understanding the nature of the materials produced for the embankment, a table of quantities has been formulated to illustrate the balance between excavation and the embankment requirements as designed and upon completion of Stage II of the contract.

Table C-1

Stage II Contract Rock Quantities

Item 7	Excavation Rock - Area A	252,000 cy	259,516 cy
Item 8	Excavation Rock - Area B	224,000 cy	242,451 cy
Item 14	Scalped Rock and Choker Course	58,000 cy	39,685 cy
Item 16	Rock Zone 1 and 2	424,000 cy	412,533 cy
Item 17	Upstream Rock Slope Protection	21,000 cy	18,612 cy
Item 18	Downstream Rock Slope Protection	15,000 cy	14,960 cy
Item 19	Channel Scalped Rock & Choker Course	33,000 cy	32,832 cy
	Stockpile #1 (Choker Course Surplus)		50,836 cy
	Stockpile #2 (Upstream Slope Protection)		27,775 cy

Total Rock Excavated 501,967 cy

Total Rock Placed 597,233 cy

Bulking Factor $\frac{597,233}{501,967} = 1.19$

Bulking Factor (Design) = 1.20

Under Item 18, Stockpiles #1 and #2 have been created as a byproduct from the manufacture of Scalped Rock and Slope Protection. This excess was not intended and schools of thought include a premise that blasting, per se, as described herein, was directly responsible for the stockpile in excess of design requirements. Appendix A has been reserved to explain in detail the "causal roots" of its existence. Item 16, Rock Zone 1 and 2, includes rock from both Areas A and B. For this reason, Item 8, Excavation Rock - Area B, has been included in the table even though no other reference is made to Area B blasting in this narrative. Bulking Factor, commonly used in achieving

an equitable balance of quantities in design, is defined as the ratio of rock placed to rock excavated. The two factors stated at the base of Table I illustrate the comparison of the design factor and the factor based on actual computations.

1-03. Geology and Petrography: The rock excavated is classified as the Cotter-Jefferson City formations, Canadian Series, of the Ordovician System. Though Cotter-Jefferson City denotes two distinct formations, the Cotter is conformable upon the underlying Jefferson City formation and bears very few distinctive differences. These formations consist of light gray, light brown to brown dolomites, moderately hard to soft, massive to very thin bedded, medium to fine grained or argillaceous, with intercalated green and black shales locally present. Cherty units affilitated with the Massive beds are common throughout. Structural anomalies identified as sink structures from the Pennsylvanian Period commonly obliterate the existing formations with breccia, sandstone, and shale intruded directly into probable sinks, caves, and cavities of the Ordovician rocks before lithification was complete. As fragmentation was the prime consideration in blasting a satisfactory rock product, a brief description of inherent fracture patterns is extracted from the MRD report. "Investigations to deterine the factors causing excessive blasting fragmentation of dolomite rock," is presented herein to illustrate the fracture frequency inherent within the rock. It is believed that the secondary fracture characteristics inherent in this rock contributed directly to the excesses of -2" rock realized during the rock blasting operations.

"Rock test samples from Area A (Calyx Holes 1 and 4) El. 616 - 665, Sample K-2, Calyx 1, El. 628 are gray, fine to medium grained conglomeratic dolomite; A face - fractures 1/2 to 1 inch intervals, B face - fractures 1/2 to 1 inch intervals. A & B faces are two sawed faces at about 90°. Sample K-3, Calyx 1, El. 652 is medium grained, gray, sucrose dolomite with much intergranular porosity; fracture interval of 1 to 2 inches. Sample K-4, Calyx 1, El. 656 is buff gray, fine grained, porous sucrose dolomite with irregular nodules of tripolitic chert, loosely cemented; A face - fractures 1/8 - 1/2 inch interval; and B face - fractures 1/8 - 1/2 inch interval. Sample K-5, Calyx 4, El. 619 is gray, fined grained, sucrose dolomite with frequent, very thin streaks of pyrite or marcasite representing bedding planes and healed fractures; A face - fractures 1/2 - 2 inch interval; and B face - fractures 1/2 - 2 inch interval.

Lifficulty in refining production blasting to reduce the amount of fines (-2 inch rock) has been compounded by the intricate fracture system inherent within the rock and the physics of detonation. The propagation of shock waves - the compressional wave emanating outward from the detonation, the tensional wave reflected back from each fracture surface encountered, and refracted shear waves; induce stresses virtually fragmenting the rock along bedding and fracture surfaces despite care in reducing the overt affects of each detonation.

1-04. Rock Breakage: In accepting the usual concept that explosives are designed to economically reduce rock ledges to a satisfactory rock product for the using agency, a peculiar concept is recognized at Kaysinger in that the

goal to blasting is to avoid excessive reduction of the rock in consideration of its soft character and intense incipient fracture pattern. These efforts have been toward blasting a coarser rock product - workable but accommodating design requirements for specific gradations. Explosives selected generally possessed a low detonation velocity, low unit weight, and a fair water resistance; a representation of an inexpensive powder that can be used in water. Low detonation velocity was selected to simulate the characteristics of impedance relating to the energy transfer from explosive to rock. Low unit weight explosives in cartridges provided for a greater resistance to water often found in drill holes below the water table. Cartridge explosives also were used to decouple the explosive from the rock. During the subject blasting operation, excepting toe charges and experimentation with HP 9, cartridge sizes were roughly 1/2 the diameter of the drill hole. This accounted for a 35 percent relative strain amplitude or that 35 percent of the explosive energy was transmitted to the rock.

1-05. Explosives:

a. Characteristics: The explosives used in Area A excavation were all cartridge type with properties described in Table C-2.

Table C-2
Explosive Properties

Brand	Weight Strength	Cartridge Strength	Rate Feet/Second	(Inches) Sensitiveness	Water Resistance	Fume Class	Number In Case
Gelamite 1	65%	60%	11,500	12	Good	1	112 - 1-1/4 x 8"
Gelamite 2	65%	45%	11,500	12	Good	1	10 - 3" x 16"
Gelamite D	60%	65%	17,700	16	Good		8 - 3" x 16"
HP 9		18%	5,520		Poor	4	10 - 3" x 24"
Hercol 4	68%	35%	9,500	8	Fair	1	26 - 2" x 16"
Hercomite 5	65%	30%	9,850	12	Fair	1	28 - 2" x 16"

Toe charges consisted of Gelamite 1, 2, or D. The Gelamites are a semi-gelating dynamite with a measure of gelatin nitroglycerines formulated to be an economical substitute for gelatin dynamites in areas where water is commonly encountered in drill holes. When maintaining proper bench heights with a minimum of toe charge became a problem, the contractor switched from Gelamite 2 to Gelamite D for the toe charge. Gelamite 1 was commonly used when small diameter charges were needed to reach the bottom of the drill hole. On other occasions, Gelamite 1 was used in drill holes bordering the final slope of Area "A" excavation to reduce the shattering affect against the backwall. Hercol 4, a low cost high ammonium nitrate explosive, was generally used as a column load. During a short period when Hercol 4 supplies were exhausted, Hercomite 5, also an ammonium nitrate explosive, served as a substitute.

b. Impedance Formula and Energy Transfer: HP 9, as reported by representatives of Hercules Powder Company, was an ammonium nitrate explosive of very low velocity formulated for the Cotter-Jefferson City formations at the Stockton Dam Project. It was specifically intended for rock with low longitudinal seismic velocities. It was theorized by representatives of Hercules Powder company that matching the characteristic impedance of the rock with an explosive of a low velocity would increase energy transfer of the explosive to the rock. Experimentation by the Bureau of Mines has realized a direct communication or energy transfer between explosives and rock with nearly matching characteristic impedance. The following definitions and calculations define the characteristic impedance of the rock and explosive.

ROCK PROPERTIES

Cotter Jefferson City Formation

Unit Weight	153 pcf
Seismic Velocity	7,000 fps

EXPLOSIVE PROPERTIES

HP 9

Unit Weight	136.5 pcf
Detonation Velocity	5,520 fps

Characteristic impedance of rock = X

$$X = (7{,}000 \text{ fps}) \frac{(153 \text{ Lb./pcf})}{(32.2 \text{ ft./sec.2})} = 33{,}300 \text{ lb. sec./ft.3}$$

X = 33,300 lb. sec./ft3 or 19.3 lb. sec./in.3

Characteristic impedance of explosive = Z

$$Z = (5{,}520 \text{ fps}) \frac{(136.5 \text{ Lb./ft.3})}{(32.2 \text{ ft./sec.}^2)} = 23{,}400 \text{ lb. sec./ft.3}$$

Z = 23,400 lb. sec./ft3 or 13.5 lb. sec./in.3

$X \neq Z$ or $19.3 > 13.5$

Though the rock at Truman Dam is the same formation blasted at Stockton Dam experimentation involving the use of HP 9 was generally considered unsuccessful in producing desired results. The theory of energy transfer based on characteristic impedance of explosive and rock is not necessarily incorrect; but too insignificant to offset the decoupling changes involving a larger diameter explosive together with greater loading density. In respect to decoupling, HP 9 cartridges measureing 3 inches in diameter transmitted 64 percent relative strain amplitude* (shock) to the rock - nearly twice as much as the 2 inch cartridges customarily used on the job. Unit weight of powder provided some measure of explosive comparison between cartridges. HP 9 cartridges 3 x 24" weighed 5 pounds, were whereas Hercol 4, 2" x 16" weighed 2 pounds. Comparatively, 1.7 times more explosive weight was used per lineal foot of powder column when HP 9 was used in preference to Hercol 4. HP 9 weighs 5 lbs./2 ft. or 2.5 lbs./l.f. of powder column and Hercol 4 weights 2 lbs./1.33' or 1.5 lbs./l.f. of powder column. It was learned that despite the characteristic impedance of the HP 9 explosive relative to the rock, considerable larger explosive force was induced by HP 9 because of its transmitted relative strain amplitude and unit weight. The reasonably good fragmentation of some shots made with HP 9 was probably due to the particular rock type, fracture configuration, and bedding thicknesses of those particular shots.

c. Decoupling: Geometrical coupling of an explosive to the rock is generally an expression of an explosive's proximity with the rock (i.e., explosive to the drill hole). Decoupling is a term used for the action in reducing that proximity with the rock. A percentage conveys the amount of coupling and, in turn, becomes a step in determining the amplitude of the explosive's strain pulse transmitted radially into the rock. This figure serves as a fraction of the total possible strain amplitude (or shock) capable of being transmitted expressed as a percent. In evaluating each shot, it was found that decoupling in water was largely ineffective as water was a good conductor of energy. A common observation of many shots indicatd that blasted thin bedded units become finely broken; whereas thick bedded or massive units became large and blocky. This could have been attributed to concentration of explosive forces in zones providing paths of least resistance to the expanding gases of detonation, though this has not been proven conclusively. A persistent problem created while practicing decoupling in the field was the cartridges slipping past each other, doubling the load per lineal foot of explosive column in several instances. When a new drill bit was introduced, hole diameter was about 4-1/4". As the bit wore down, the problem of cartridge slideby rectified itself until another new bit was used. Then problems in cartridge slideby recurred.

* See Relative Strain Amplitude Plotted Against Coupling (Slightly Modified from Atchison, T.C., 1961, Effect of Coupling on Explosive Performance; Quarterly, Colorado School of Mines, Vol. 56, No. 1).

The contractor initiated three attempts to alleviate the situation. On Shot 3, lift 1, each cartridge was strung with 50 grain primacord to prevent the cartridges from passing each other. To the dismay of the contractor, considerable more time and effort was required in loading. The shot was classed as a failure because primacord running from the charges "down the hole" to the surface negated the stemming and permitted the expanding gases to the vent. Fragmentation was poor. The rock product consisted of several large size boulders requiring additional effort in redrilling and reblasting. Secondly, a 3-1/2 inch bit was used to remedy the problem. This approach was partially successful except that the relative strain amplitude increased from 35 percent to 51 percent. This approach was ultimately abandoned as considerable time and difficulty was experienced in situating the toe charges, customarily 3 inch diameter cartridges. It was found that the problem was alleviated somewhat by alternating the 2 inch cartridges of Hercol 4 with 3 inch cartridges of HP 9. This, however, increased the density of powder per lineal foot of column and also the powder factor in many cases. Objections to the use of HP 9 for this operation were the same as those against the exclusive use of HP 9 as a column load.

1-06. Blast Holes:

a. Hole depth: In the field, hole depth was fixed as required by the specifications. Two feet of subdrilling was permitted to allow the holes to achieve a depth equal to the bench height plus two feet. Control was necessary in attaining this depth since the surface was not level. Carelessness on the part of the drillers also provided some variation.

b. Hole diameter: Hole diameter was generally discussed in detail under paragraph 1-05 (c).

c. Quality of drilling workmanship: At the beginning of the work, considerable difficulty was experienced in maintaining clean holes as the contractor failed to protect drilled holes. When efforts were required to redrill holes to clear them of mud, water, and debris, the contractor proceeded to protect the drilled holes. The presence of water posed a persistent problem, requiring the need of water resistant cartridge explosives. Water ranged in depth from two to eight feet as a rule, to full holes on occasion. Decoupling was nullified somewhat, because of these conditions.

d. Spacing - burden: Spacing - burden was varied early in the work to determine the more satisfactory hole configuration. To gain from the experimental blasting performed under direction of Mr. Robert Stansfield, Geologist, KCDO, 21 October to 26 November 1966, it was suggested to the contractor that spacing-burden ratio of 2 or greater would produce more acceptable results. Therefore, the ultimate spacing-burden ratio became 14 x 7 to accommodate the theory and permit the drillers to remain abreast of the rock blasting and excavation. Several discussions were held with the contractor in arriving at this figure and expanded variations of it. The contractor was hesitant about expanding the distance on either burden or

spacing because of the possibility of failing to blast a level floor or "pulling the shot." Since powder factors were approaching that suggested in the specifications, little success was made in unseating the contractor's position. When work began on the second lift, a 15 foot bench, the burden-spacing inadvertently remained the same as did the contractor's initial approach to stemming. Consequently, the powder column became smaller relative to the rock required to be removed by each loaded drill hole, and the powder factor was subsequently reduced to less than 0.3 pounds per cubic yard in several cases. With some difficulty in rock excavation and pulling the floor, the rock was successfully removed with a 4.5 cubic yard Northwest shovel. The muck pile hardly represented a fragmented rock, since a shot of this powder factor merely cracked the rock without measurable throw. The problem of achieving an ideal minimum of fines continued despite the "tender loving care" approach to blasting. It became readily apparent that the shovel was able to remove the rock without its being blasted. Continuing this approach to rock excavation was discouraged because the shovel would ramp up from the floor creating a toe requiring removal by ripping. Thus, after each shot was made, markers were situated along the limits of the shot defining these limits for the shovel operator.

e. Stemming: Oversize rock became a problem creating difficulty in excavating the muck pile and requiring the contractor to provide a near full time operation in separating oversize pieces and breaking them down to workable sizes. Since the burden-spacing remained relatively fixed, stemming served as the tool in varying powder factors in accomplishing the work. As it was discovered, the rock could be removed with powder factors well below 0.5 pound per cubic yard and the normal principles of stemming nearly equal to the burden no longer applied. Stemming reached a depth of ten feet on scattered occasions compared to a burden of seven feet. Thus, oversize rock was developed from this zone. In trying to eliminate the oversize problem, stemming was varied from hole to hole in each row. Depth of stemming figures often used were 7 and 11 alternating respectively in each hole in row. This provided some measure of success in reducing the oversize problem. The attitude toward stemming has been nurtured by the observation of progressive shots in Area "A" and the disposition of the particles in respect to size. Some significant observations have concluded that the fines noted were situated throughout the base of the shot, whereas coarser fractions have rested on the surface. Every shot had the appearance of a good quality product until excavation began. Amount of fines increased as the excavation advanced toward the toe of the bench. Increasing the amount of stemming to reduce fines transpired from this observation. Physically, it appeared that the stemming produced the coarser fraction and the powder column created the finer fraction. Smaller diameter holes, smaller size cartridges, and smaller burden spacing was suggested to the contractor at one time to reduce the fracture effects of the existing powder column to something with less explosive force. This suggestion was too radical for the contractor to readily adopt as he feared failure in removing the shot without redrilling or in developing a toe in the floor of the excavation.

f. Delays: It has been found that electric caps delayed row on row also contributed to oversize rock. Each row fired together provided a combined force uniformily outward toward the face creating a prominent crack between rows. Occasionally detonation failed to satisfactorily crack the rock normal to the rows in the zone of stemming. Consequently, it was decided that if delays were varied in row, the energy would not combine to create this somewhat larger force of detonation. The alternating blasts would tear the rock creating strain cracks normal to the rows and reduce the effort parallel to the rows. Earlier, in developing the blasting program, powder factors were sufficiently high to permit the rock to throw. During this time, it was theorized that rock in motion tended to reduce in gradation merely by particle collision. The contractor reduced the number of rows per blast from five or four rows to three or two rows. This in itself proved successful in reducing the fines, though more individual shots were necessary to remain abreast of the shovel excavation. This approach was abandoned after powder factors were reduced below the level permitting rock throw.

g. Powder factor is defined as the pounds of explosive per cubic yards of rock. Yield was calculated in two ways to base a comparison of powder factors on each shot. Yield was initially calculated after measuring the volume of rock to be removed. Yield was again calculated on the number of holes multiplied by the amount of rock removed by each hole. Theoretically, the powder factors should be equal. Actually, the difference measures the accuracy of the drilling pattern. Drillers without proper supervision often were discovered to guess on hole location. When powder factors failed to accurately agree, it was often found that spacing between holes and between rows were inaccurate. At other times, it was found that additional holes were often drilled because ragged backbreak indicated that additional holes were necessary. Nevertheless, the irregular pinpointing of the extra holes was reflectd in the difference in powder factors. In isolated cases, this approach helped to realize the existence of a rock toe or a reduction in bench height.

Manipulation of the explosive figures helped to recognize the problem of "cartridge slideby" in the drill holes. The number of cartridges required for each drill hole was easily calculated. The numer of cartridges actually used was readily determined as well. The difference between the theoretical number of cartridges to be used in the shot and the number of cartridges actually used generally reflected the number of cartridges by-passing each other in the hole, since depth of stemming was the principle guide to loading the holes in the field.

These two principles served to represent the degree of refinement the contractor was applying toward his blasting operation.

1-07. Rock Gradation Tests: During the course of excavating Area "A", eight gradations on the rock product were periodically made as required by the specifications. These gradations provided a systematic gage in ascertaining progress in reducing the percentage of fines. Each gradation was used to demonstrate a key feature in attempting to achieve this end.

Reference is made to Table C-3, listing the gradations chronologically by shot and lift. The following statements provide a brief explanation of each shot reflecting the results stated.

Table C-3

ROCK GRADATIONS - STAGE II

Percent Passing — Area A

	Description	36"	30"	24"	18"	12"	10"	8"	6"	4"	2"
1	Shot 2 Lift 1	100	100	94.0	85.9	73.8	72.9	67.3	56.8	44.8	28.3
2	Shot 3 Lift 1	100	100	88.6	87.9	85.3	81.0	73.2	65.5	55.2	34.9
3	Shot 6 Lift 1	100	100	100	95.9	89.6	84.6	79.7	70.8	60.8	42.0
4	Shot 16 Lift 1	100	92.3	85.8	81.7	65.6	59.2	54.2	47.7	39.7	26.2
5	Shot 2 Lift 2	100	97.5	96.4	93.0	86.9	81.7	75.3	66.5	56.6	38.3
6	Shot 4 Lift 2	100	100	96.3	92.7	87.2	84.4	80.3	74.3	64.6	46.8
7	Shot 13 Lift 2	100	97.4	95.2	86.9	65.8	59.7	53.0	43.0	34.0	20.6
8	Shot 22 Lift 3	100	94.9	94.9	86.7	80.3	74.9	68.8	60.7	51.2	34.2

a. Gradation 1, Shot 2, Lift 1: Care was exercised in assuring the workability of the decoupling principle by introducing polystyrene paper atop each powder column to prevent stemming material from drifting alongside the cartridges and inhibiting the decoupling effect. The powder factor was higher than desired (0.69). Because of this, it was believed that the burden spacing could be expanded. Cartridges of explosives by-passing each other created a situation leading to an increase in powder factor as well. The fines were markedly low (28.3 percent) inspite of the adverse factors affecting fragmentation. It is believed that prominent thicker beds may have advantageously aided in keeping the fines lower than expected.

b. Gradation 2, Shot 3, Lift 1: This shot was loaded with meticulous care to achieve maximum utilization of the decoupling theory. Each cartridge in the column load was threaded with 50 grain primacord to assure against cartridges slipping past each other. The powder factor was less (0.56) than Shot 2 by 0.13 lbs/c.f. essentially because of this measure. The gradation results indicated a finer fragmentation; however, there were a large number of oversize blocks that failed to break down. Had oversize been included in the gradation, the end results would have been coarser. A critical problem on oversize was created by this approach. Bedrock on the bench blasted could be classed as thick bedded to massive and the 50 grain primacord used to thread the cartridges extended to the surface through the stemming. It is believed that the primacord detonation through the stemming provided the release of expanding gases of the explosive detonation, thus reducing the explosive force. This approach to blasting was subsequently dropped because of the expended loading time and the undesirable accumulation of oversize rock.

c. Gradation 3, Shot 6, Lift 1: Column load for this shot was HP 9, an explosive with a low detonating velocity, to exercise the relationship between the characteristic impedances between the rock and explosives. The actual powder factor (0.81) demonstrated the fact that density per lineal foot of powder column was higher in this case and that the burden-spacing (8x12) was not expanded satisfactorily in achieving a sufficient load. The fines created demonstrated these facts.

d. Gradation 4, Shot 16, Lift 1: In an effort to relieve the problem of cartridges slipping past each other and utilize the HP 9; Hercol 4 and HP 9 were alternated in the powder column dropping the powder factor to 0.45 Lbs/c.y. Stemming equaled burden. Since this was a corner shot, throw was minimized. Fines were held to a minimum and concentrated at the base of the bench presumably matching the powder column.

e. Gradation 5, Shot 2, Lift 2: This shot was confined laterally on all four sides. Consequently, burden spacing was reduced to 7'x 11' and powder factor raised to 0.72 to assure adequate fragmentation to the proper depth. The rock product appeared satisfactory. Throw was contained, particle collision reduced, and fines were supposedly held to a minimum. Gradation results proved differently, however. A higher percentage of fines than expected was realized indicating that the merits of confined shots could not be substantiated.

f. Gradation 6, Shot 4, Lift 2: When bench height is reduced; burden-spacing, stemming, and loading techniques remain constant, the height of the powder column becomes smaller, and the powder factor is reduced. On this shot the powder factor was reduced to 0.28. Thin to medium beds prevailed. Very little rock disturbance was affected or throw produced. The fragmentation was unusually high. It is believed that the rock's soft properties, bedding character, incipient fracture pattern, and the close proximity of the rock to the powder column contributed measureably to the production of excessive fines.

g. Gradation 7, Shot 13, Lift 2: Loading methods had not changed markedly since the gradtion test on Shot 4 was made. Shot rock was coarser by 26.2 percent on the two-inch screen, however. It is an obvious assumption that bedding directly contributes to the fragmentation of the rock product. A predominance of rock with thick or massive beds yields less fines than a bench with considerable thin beds and shale seams.

h. Gradation 8, Shot 22, Lift 3: This shot had implemented the most recent adopted principles affecting the reduction of rock fines such as alternating the stemming in each hole, alternating the delays in each hole in the same row and maintaining a low powder factor (0.35 lb./c.y.). Since the rock product appeared coarse and well graded, the results actually reflected finer rock than expected. It is believed that the fines were produced directly from the rock matching the powder column, and thin bedded characteristics more common in this bench.

1-08. Conclusions: There are an infinite variety of approaches that remain to be examined; however, feasibility of exploiting several avenues during production blasting became limited at best. As a guide, the most applicable reference to date has been the report on Test Blasting at Kaysinger Bluff Dam, by Robert Stansfield, KCDO. This report has served the geologist in encouraging the contractor toward adopting improved methods to blasting the Cotter-Jefferson City Formation to desirable sizes. Some success was made in guiding the contractor but greater cooperation could have induced inmeasurable improvements, especially in respect to expanding burden spacing and reducing cartridge slideby. It is also believed that rock quality has not been fully evaluated in recognizing difficulties in achieving particular gradation on large quantities of rock. The two large stockpiles of rock residue serve as a a monument to rock quality and design considerations. From what is now known, it is possible, or even probable, that the most applicable combination of blasting methods would not have avoided excesses in fines (-2" rock) because of the rock properties (e.g., softness, presence of incipient fractures, thin beds and the presence of shale).

SUPPLEMENT D

SHOT RECORDS POWERHOUSE EXCAVATION

1-01. Shot Records:

a. General: The tabulated tables reflect a synopsis of shots of the powerhouse structure area with the exception of ramp, sump, and rim shots of minimal scope. This tabulation is devised to punctuate the factors of explosives, rock, and loading for easy identification and ready comparison. The relationships as abbreviated in this tabulation were extracted from the field notes and compiled to document the findings described in Supplement C. Each shot is numbered consecutively by lift beginning at the top of rock. Yield, Explosives, Rows, and Number of Holes are self-explanatory. Burden-Spacing, Bench Height, Hole Depth, and Stemming are expressed in feet. Column Load and Toe Charge refer to number of cartridges. Delay refers to electrical millisecond delays in this respective order. Instants, 25, 50, 75, or 100 millisecond delays are labelled numerically as 0, 1, 2, 3 and 4.

b. Powder Factor: In order to fully evaluate the powder factors, it is necessary to understand the definition of Theoretical Powder Factor (PF) and Actual Powder Factor (PF).

$$\text{Theoretical PF} = \frac{E}{N \times N_{cy}}$$

Where E is total amount of explosives used in the shot
N is total number of holes detonated in the shot
N_{cy} is number of cubic yards of rock represented by each hole.

$$\text{Actual PF} = \frac{E}{cy}$$

Where E is total amount of explosives used in the shot
cy is the total cubic yards of rock shot as physically measured in the field before detonation.

The purpose of distinguishing the two powder factors as described hereinabove was to provide a tool in determining the accuracy of workmanship in laying out the hole pattern, drilling, and loading the shot. A wide difference between the Actual and Theoretical Powder Factors indicated considerable diversity in the actual pattern of drilling opposed to the intended order of the shot. The application of this tool induced a quest for determinants creating the variance. In most cases, there was sufficient time to realize the difference between powder factors and explore the reasons for this difference during the time after loading of the shot was completed but before detonation. In this way, continued pressure could be applied on the contractor in his improving the accuracy of the hole pattern layout, drilling, and loading. Particular problems such as explosive cartridges slipping past each other in the drill hole were often indicated by the difference in the powder factors. It was soon realized that a mathematical tool has been utilized in improving quality of workmanship in the field.

c. <u>Blasting Ratios</u> represent numberical relationships based on burden. These blasting ratios proposed by Professor Richard Ash, The University of Missouri - Rolla, have relegated relationships in blasting to figures for easier statistical comparisons. Frequency distributions have shown particular figures commensurate with desired results. It is believed that the Blasting Ratios as presented herein are insufficient in quantity to establish a trend; however, before the project is completed, there may be a sufficient number of figures to permit definite blasting ratios for determining desired results. Upon completion of all blasting at Truman Dam, the Blasting Ratios will be combined to include the results of all the shots in the Powerhouse Structure Area. In the meantime, the data has little real value except in comparing the Blasting Ratios with the table of all types of blasts published by Professor Ash in Pit and Quarry Magazine, Vol 56, Nos. 2, 3, 4, and 5, reproduced on the following page.

Table D-1

<u>Standard Blasting Ratios for Vertical Blastholes</u>
(All Types of Surface Blasting, 20 Different Rock Types, Hole Depths From 5 to 260 ft., and Hole Diameters From 1-5/8 to 10-5/8 in. for all Grades of Explosives)

All Operations				All Operations but Coal Strippings			
K_B Group	Frequency	K_H Group	Frequency	K_J Group	Frequency	K_T Group	Frequency
						0.10-0.19	0
		0.0-0.9	0			0.20-0.29	6
10-13	0	1.0-1.9	43			0.30-0.39	12
14-17	5	2.0-2.9	70	0.00-0.09	15	0.40-0.49	18
18-21	13	3.0-3.9	56	0.10-0.19	18	0.50-0.59	18
22-25	51	4.0-4.9	45	0.20-0.29	27	0.60-0.69	25
26-29	74	5.0-5.9	22	0.30-0.39	26	0.70-0.79	19
30-33	66	6.0-6.9	22	0.40-0.49	25	0.80-0.89	13
4-37	44	7.0-7.9	11	0.50-0.59	2	0.90-0.99	6
38-41	20	8.0-8.9	4	0.60-0.69	6	1.00-1.09	14
42-45	7	9.0-9.9	2	0.70-0.79	2	1.10-1.19	7
46-49	4	10.0-10.9	8	0.80-0.89	0	1.20-1.29	7
50-53	0	11.0-11.9	0			1.30-1.39	3
		12.0-12.9	1			1.40-1.49	2
						1.50-1.59	2
Total	284	Total	284	Total	125	Total	152
Mean	30	Mean	4.0	Mean	0.28	Mean	0.74
Mode	38	Mode	2.6	Mode	0.24	Mode	0.65
Median	29	Median	3.4	Median	0.27	Median	0.67

Table D-2

SEE PLATE NO. 166

in Volume Two of this report

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